December 2005

Detailed Design Report

Dade County Beach Erosion Control and Hurricane Protection Project

Bal Harbour Segment

Dade County, Florida

US Army Corps of Engineers
Jacksonville District

DETAILED DESIGN REPORT
DADE COUNTY BEACH EROSION CONTROL & HURRICANE PROTECTION PROJECT
BAL HARBOUR SEGMENT

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT'D)

LIST OF TABLES

TABLE OF CONTENTS (CONT'D)

LIST OF FIGURES

APPENDICES

DETAILED DESIGN REPORT

DADE COUNTY BEACH EROSION CONTROL & HURRICANE PROTECTION PROJECT, BAL HARBOUR SEGMENT

INTRODUCTION

This report examines the performance of the Bal Harbour segment of the Federal Beach Erosion Control and Hurricane Protection Project at Dade County, Florida. Specifically, this report will analyze erosional areas along the Bal Harbour shoreline and determine the feasibility of providing additional shore protection measures to reduce rapid losses of beach fill along this segment of the project. Data gathered during the physical monitoring of the project will be examined in order to quantify physical processes within the project area, and to suggest methods for managing the project in the most economically efficient manner possible.

Authority.

The Beach Erosion Control and Hurricane Protection Project (BEC & HP) for Dade County, Florida was authorized by the Flood Control Act of 1968. In addition, Section 69 of the 1974 Water Resources Development Act (Public Law 93-251) included the initial construction by non-Federal interests of the 0.85-mile segment along Bal Harbour Village, immediately south of Bakers Haulover Inlet. The authorized project, as described in House Document 335/90/2 (reference 1a), provided for the construction of a protective and recreational beach and a protective dune for 9.3 miles of shoreline between Government Cut and Bakers Haulover Inlet (encompassing Miami Beach, Surfside, and Bal Harbour), and for the construction of a protective and recreational beach along the 1.4 miles of shoreline at Haulover Beach Park.

The Beach Erosion Control and Hurricane Protection Project for Dade County, Florida, North of Haulover Beach Park was authorized by the Supplemental Appropriations Act of 1985 and the Water Resources Development Act (Public Law 99-662) of 1986. This authorization provides for modification of the authorized 1968 Beach Erosion Control and Hurricane Protection Project for Dade County, Florida, to provide for the following:

a) The construction of a protective beach along a reach of shoreline extending 2.4 miles through Sunny Isles, and for periodic nourishment of this area.

b) The extension of the period of Federal participation in the cost of renourishing the existing Dade County Beach Erosion Control and Hurricane Project from 10 years to 50 years.

Purpose and Scope.

The purpose of this report is to evaluate project performance along the length of Bal Harbour, and to examine methods of improving project performance. Physical data will be analyzed and numerical shoreline modeling will be performed in order to identify any areas of excessive erosion within the limits of the Federal project, and to determine the processes responsible for causing erosion. Methods of reducing erosional losses will then be examined in this report, including the construction of additional structural and non-structural shore protection features. Specific structural measures to be investigated include the use of groins, revetments, and offshore breakwaters. Non-structural beach fill alternatives to be investigated include the construction of varying berm widths, beach fill transitions, perched beaches, and submerged berms. Detailed design of selected alternatives will be performed in this report. The selected design will be the basis for the creation of plans and specifications for construction of the recommended improvements.

A secondary purpose of this report is to develop sources of borrow material capable of sustaining the Bal Harbour shoreline throughout the life of the project. Dade County's offshore borrow areas have been nearly depleted, and new sources of material are required in order to ensure that the project can be maintained throughout the remaining 29 years of the project life, and beyond. An analysis of physical data, construction costs, and environmental issues for potential borrow sources will be used as a basis of determining which areas are most favorable for future use.

PROJECT AREA

Project Location.

Dade County is located along the southeast coast of Florida, and contains the city of Miami. Broward County (Ft Lauderdale) lies to the north, and Monroe County (Florida Keys) lies to the south of Dade County. The Dade County shoreline extends along two long peninsular barrier island segments and three smaller islands, each of which is separated from the mainland by Biscayne Bay. The city of Miami is located on the mainland, and a number of coastal communities are located along the barrier islands. These barrier islands vary in width from about 0.2 to 1.5 miles, with an average width of about 0.5 miles. Elevations along the entire coastal region (and much of the mainland) are low, generally less than 10 feet. Along the coastal region elevations are generally the highest along the coastline, sloping gradually downward toward the bay.

Bal Harbour (formally known as Bal Harbour Village) is located on the southernmost peninsular barrier island in Dade County. This island is bounded by Bakers Haulover Inlet to the north and Government Cut to the south, and contains the communities of (proceeding from north to south) Bal Harbour, Surfside, and Miami Beach. The Bal Harbour segment of the Federal project extends along the entire 0.85-mile length of the town's Atlantic shoreline. This reach of shoreline is fully developed, primarily with oceanfront hotels and condominiums. A project map is shown in figure 1, and an aerial photograph of the Bal Harbour study area is shown in figure 2.

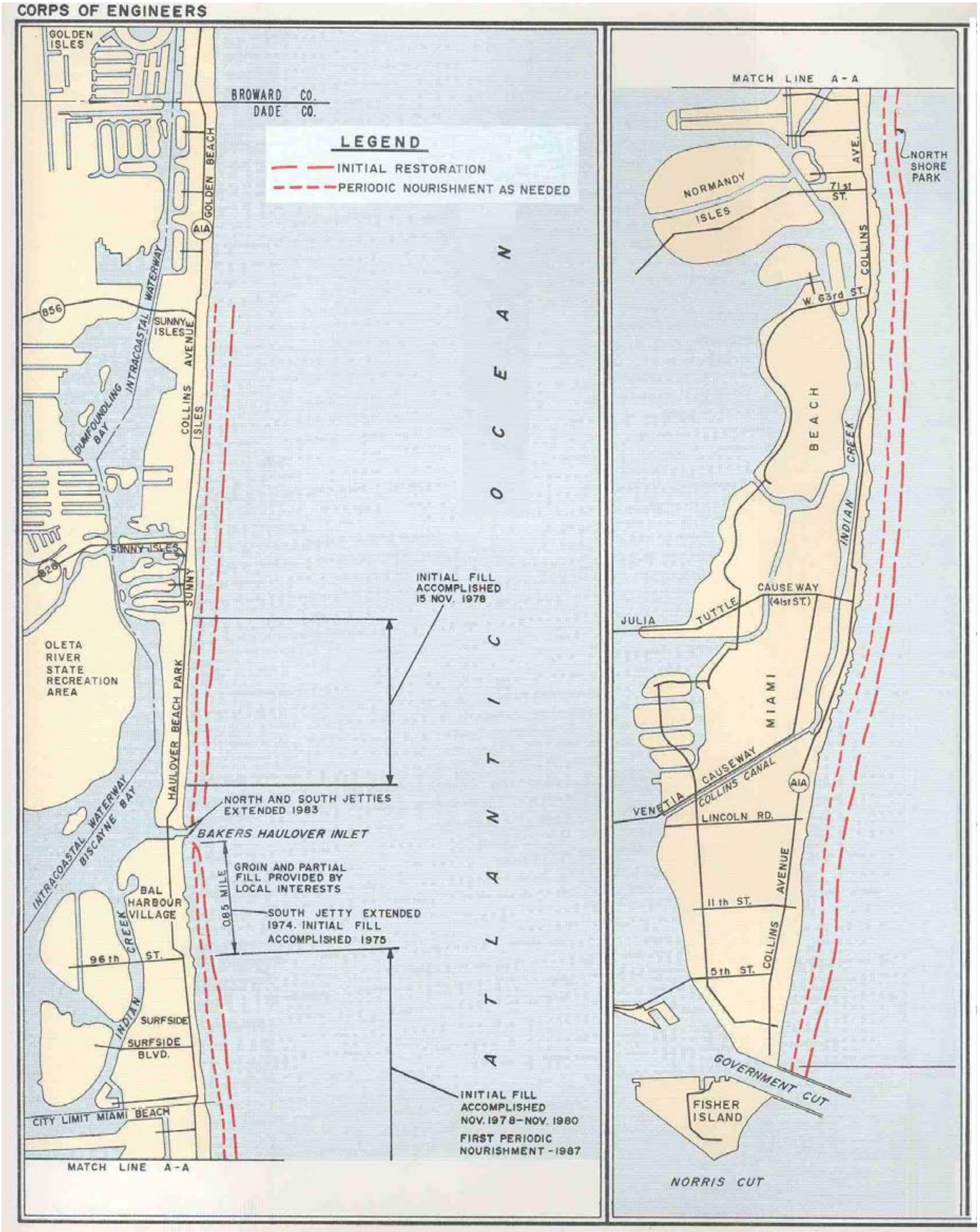


Figure 1 : Study Area; Project Map

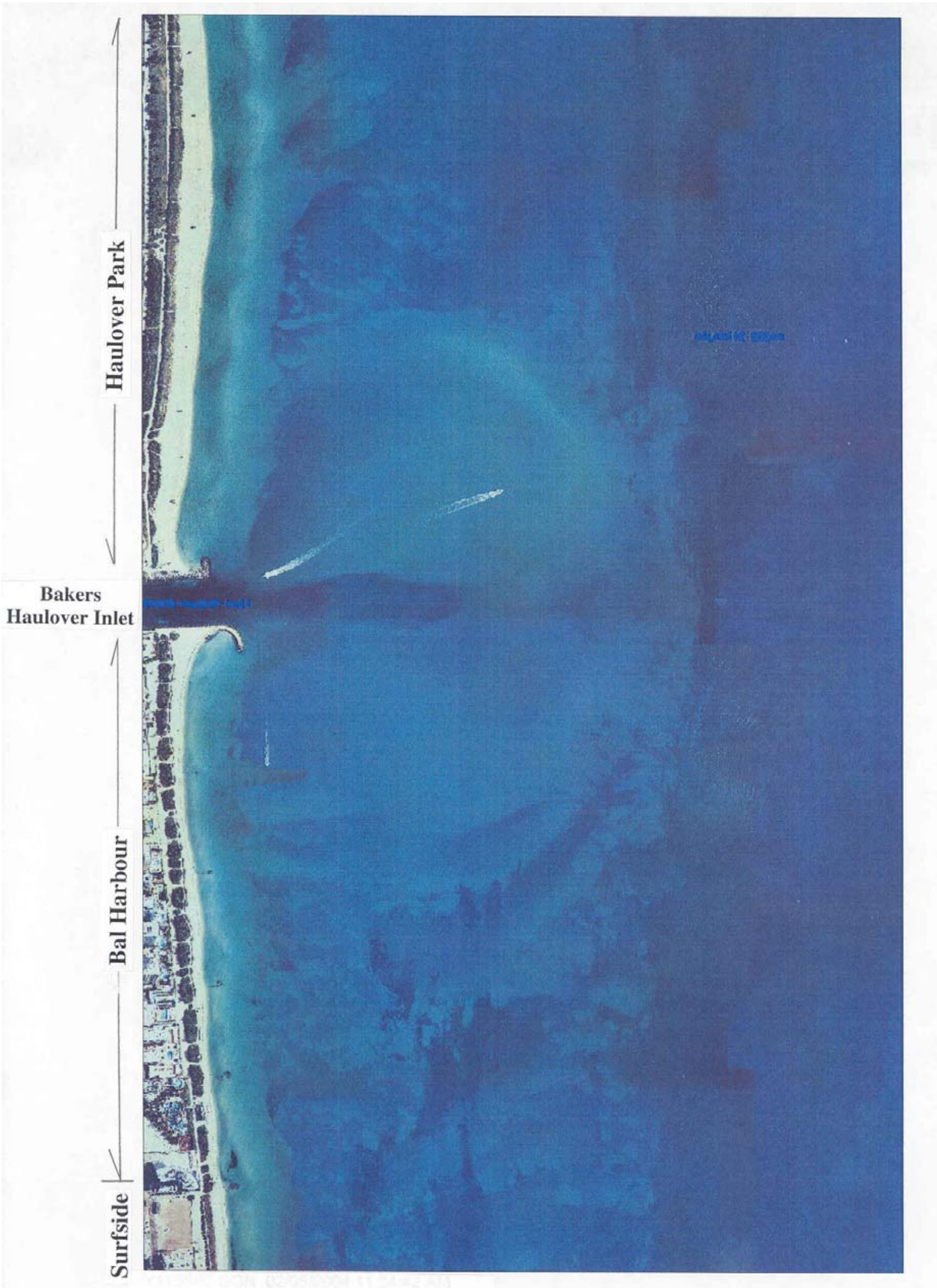


Figure 2. Aerial Photograph of Bal Harbour Study Area.

Pre-Project Conditions.

Prior to the construction of the Federal BEC & HP project in the late 1970's, substantial development of the Dade County shoreline was already well underway and much of this development had an impact on the existing coastal processes. By the 1930's, seawalls had been constructed along most of the length of the county's oceanfront to protect upland development, including along the length of Bal Harbour. In 1927 and 1930, a large number of groins were constructed along Miami Beach as part of a protective-works project at that time. The structures constructed in 1927 were typically 200 feet long and 300 feet apart, while the groins built in 1930 were typically 170 feet long and 250 feet apart. Both sets of groins were constructed of steel sheet-pile and were cross-braced with timber. Throughout the 1940's, 50's and 60's construction of large numbers of additional groins occurred along the remaining length of the county.

A continuous groin field extended along most of the length of the Dade County shoreline prior to construction of the Federal beach restoration project, which began in the late 1970's. Most of these structures remain in place today, buried by the existing Federal project. Five relic king-pile groins can be seen along the length of Bal Harbour as the beach fill recedes between project renourishments. Renovation of these groins will be examined in this report as one of several alternative plans of improvement.

EXISTING FEDERAL PROJECT

Description of Project.

The Federal project extends along the entire length of the two peninsular barrier islands in Dade County (excluding the Town of Golden Beach), covering 13.2 miles of shoreline. Three different design beach fill cross-sections are provided along the Dade County BEC & HP project, corresponding to different levels of protection along different reaches of the project shoreline. The baseline for the Federal project is the Erosion Control Line (ECL), which extends along the length of both barrier islands. The ECL approximates the position of the pre-project mean high water line.

For the 9.3–mile reach extending from Bakers Haulover Inlet southward to the north jetty at Government Cut (encompassing the communities of Bal Harbour, Surfside, and Miami Beach), the primary purpose of the project is to provide storm damage protection to the upland development, and to provide protection against hurricane tidal flooding. The design cross-section along this reach of the project consists of a 45-foot wide dune constructed seaward of the ECL, with a dune crest width of 20 feet at elevation 11.5 feet, referenced to mean low water (mlw). A 50-foot wide berm at +9.0 feet mlw elevation extends seaward from the toe of this dune. The front slopes of the beach face are 1v : 20h from the seaward edge of the berm to mlw, and 1v : 40h from mlw to the intersection with the existing bottom. Dune vegetation was planted on the slopes and crest of the storm dune along the entire length of this 9.3-mile reach of shoreline, to enhance dune stability.

The source of all fill material for construction of the Dade County BEC & HP project was a series of offshore borrow areas, shown in figure 3. These borrow areas are located along deposits of beach-quality sand which lie between the shore-parallel coral reefs offshore of the Dade County shoreline. Pre-project depths in these borrow areas were generally about 40 to 70 feet.

Other Nearby Federal Projects.

Other than the Haulover Beach Park and Sunny Isles segments of the Federal BEC & HP project to the north, the only other nearby Federal project which may have any influence on the Bal Harbour shoreline is the Federal navigation project at Bakers Haulover Inlet, located immediately north of Bal Harbour. Bakers Haulover Inlet is a manmade cut across the peninsula between the present-day locations of Bal Harbour and Haulover Park, and was constructed by local interests in 1925. At the time of construction the cut through the peninsula was about 1,100 feet long, and the inlet was about 300 feet wide (ocean side) by 500 feet wide (bay side), with a controlling depth of about 14 feet. The Federal project provided for rebuilding the north jetty and constructing a revetment along the north bank of the inlet (completed November 1963), and for constructing the south jetty and the revetment along the south bank of the inlet (completed July 1974). The Federally-authorized channel is 11 feet deep, mlw, and 200 feet wide through the ocean entrance, and 8 feet deep by 100 feet wide from the entrance channel to the Intracoastal Waterway. A project map of the Bakers Haulover Inlet navigation project is shown in figure 4.

The effects of Bakers Haulover Inlet on the Bal Harbour project will be examined in detail in this report. Although the authorized channel depth is 11 feet mlw, actual depths through Bakers Haulover Inlet are typically much deeper due to tidal current scouring. The most severely scoured areas are located through the throat of the inlet, extending seaward past the ebb shoal. Depths in these areas can exceed 20 feet, and may present a significant barrier to sediment transport along the coast. Additionally, tidal currents through the pass may directly affect the northern portion of the Bal Harbour project, and the presence of the large ebb shoal and high ebb current velocities may adversely affect the Bal Harbour beaches through wave energy focusing.

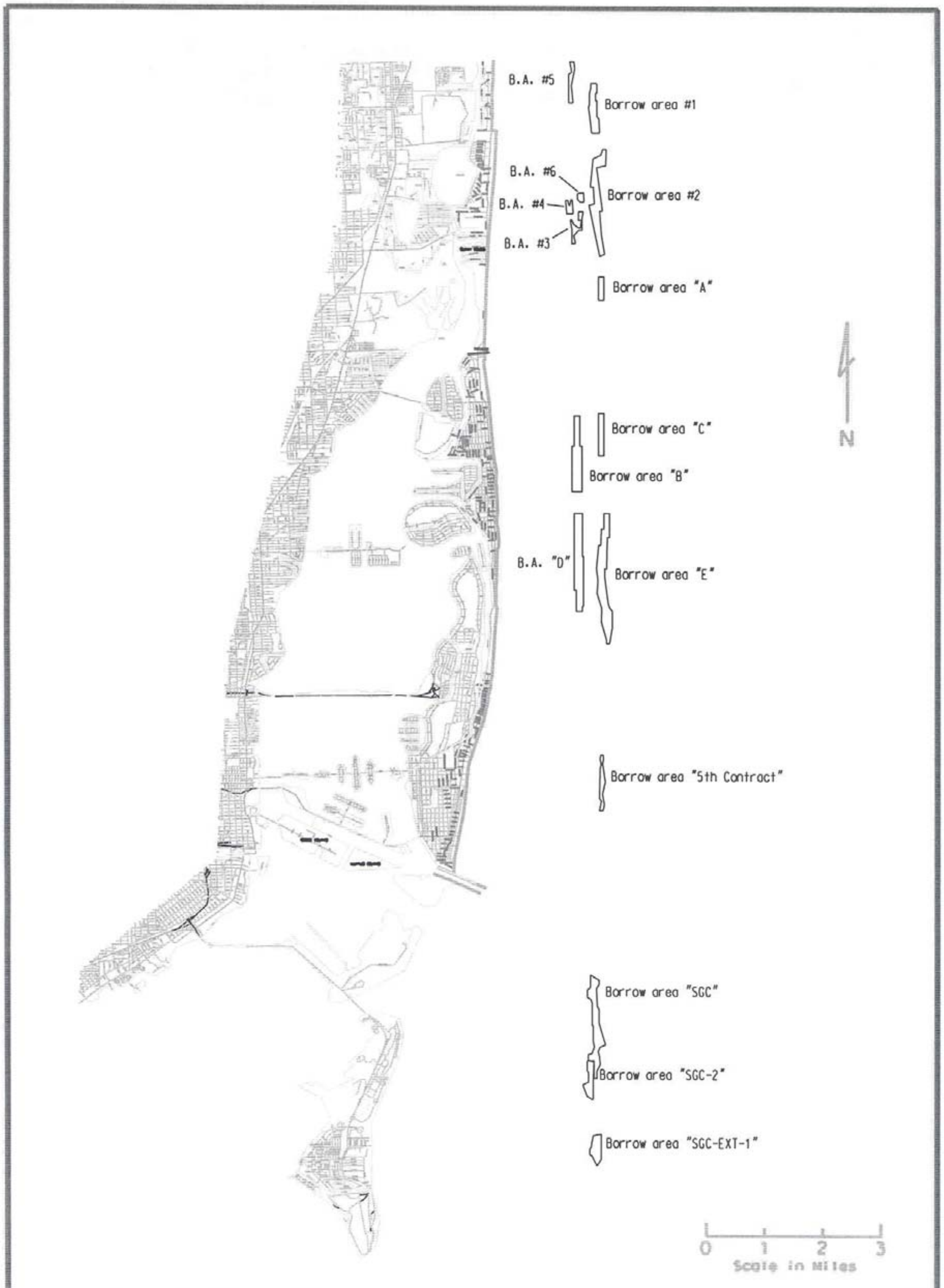


Figure 3. Dade County BEC & HP - Offshore Borrow Areas.

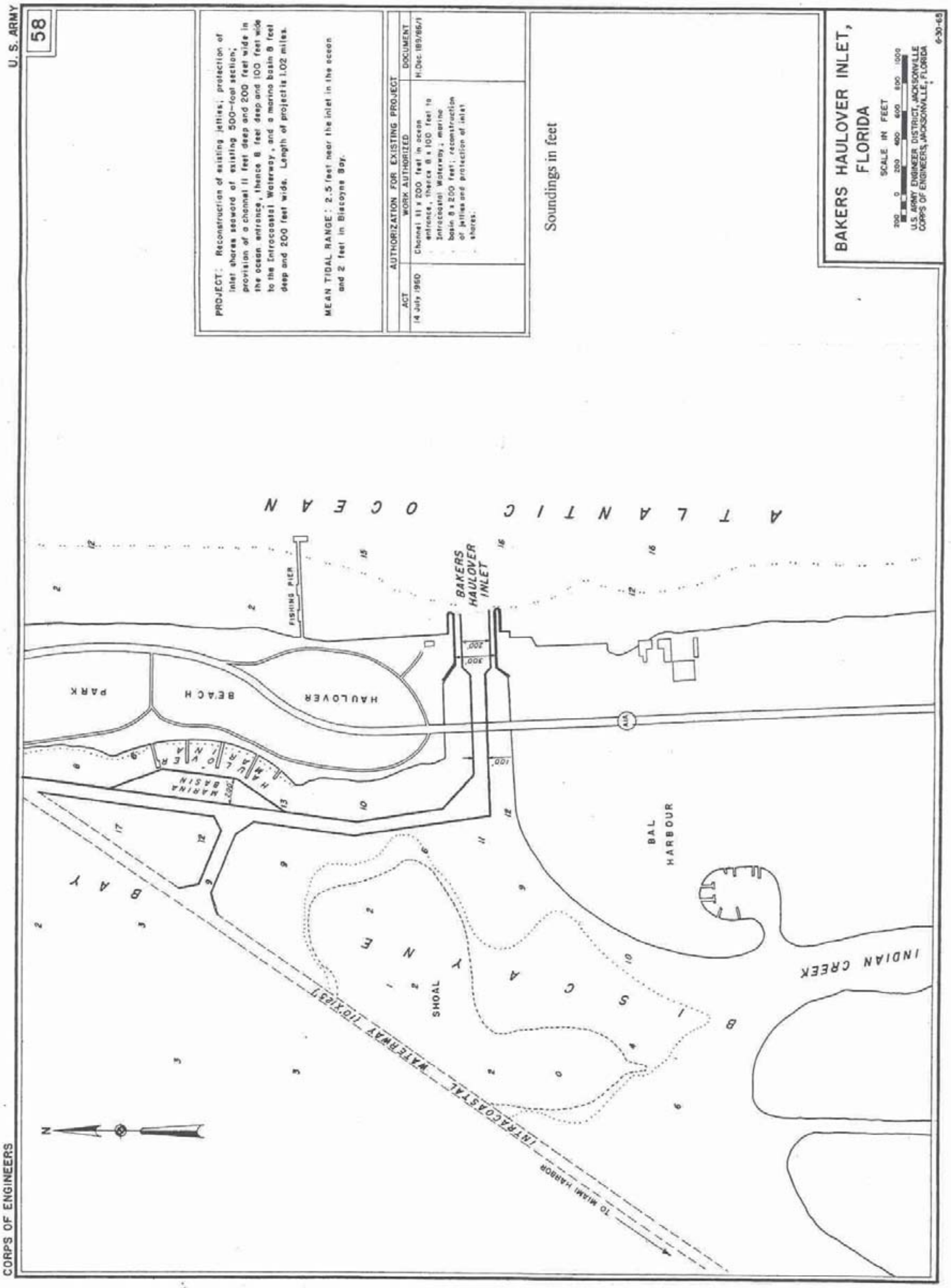


Figure 4. Project Map - Bakers Haulover Inlet Federal Navigation Project

PROJECT HISTORY

General.

The project as originally authorized provided for the placement of beach fill along the 9.3-mile reach of shoreline extending from Bakers Haulover Inlet to Government Cut, and along the 1.4-mile length of Haulover Beach Park, located immediately north of Bakers Haulover Inlet. The 2.4-mile length of Sunny Isles was added to the project in 1985 under a separate authorization. Work on the project (as originally authorized) began in 1975 and was completed in January 1982 at a total contract cost of about $48 million. Due to the length of shoreline involved, the project was constructed in several phases with each phase being administered under a separate contract.

The following paragraphs provide a chronological description of each phase of the initial construction of the Dade County BEC & HP project. Particular detail is provided for construction events along Bal Harbour and the adjacent communities, but brief discussions of all other project segments will be included in this section in order to present a complete sequence of events relating to the construction and maintenance of the Dade County BEC & HP project. A more detailed history of the remaining segments of the project is provided in the Dade County BEC & HP project Evaluation Report, reference 1g.

In addition to beach fill placements, other project-related structural improvements will be discussed, such as modifications to the navigation jetties at Bakers Haulover Inlet and Government Cut, construction of a series of detached breakwaters at Sunny Isles, and construction of shore-connected breakwaters at Miami Beach.

Initial Project Construction.

In July 1975, initial construction of the Federal project began with the placement of 1,625,000 cubic yards of fill along the 0.85-mile length of Bal Harbour. The construction berm width along Bal Harbour was 240 feet from the ECL at an elevation of +9 feet mlw, extending from the north city limit at the Bakers Haulover Inlet south jetty southward to the Bal Harbour/Surfside city limit at 96th Street. These limits of construction correspond approximately to DNR survey monuments R-27 to R-31.5. The construction template included the storm dune feature previously described. The south jetty at Bakers Haulover Inlet was extended about 735 feet with a southerly curving end section at the same time as the beach fill construction. Both projects were funded locally; upon completion the Federal portion of project costs were reimbursed by the Jacksonville District, Corps of Engineers.

In 1978, Bal Harbour planted hundreds of coconut palms along with sea oats, sea grapes, and other assorted dune vegetation along the upper portion of the project berm along the entire 0.85-mile length of the city’s oceanfront. This work was not within the scope of the Federal project, and was funded entirely by the city of Bal Harbour. The vegetation not only greatly enhances the aesthetics of the beach, but also provides shade to beachgoers and provides stabilization against wind erosion.

Because of the project size, the remaining 9.8 miles of shoreline was divided into 5 segments or phases, each to be constructed under a separate contract. The sequence of construction was from north to south, excluding Bal Harbour, which had been constructed several years earlier.

The Phase I contract provided for fill placement along two reaches of shoreline. The north fill area (300,000 cubic yards) extended along a 1.1-mile reach of Haulover Beach Park, immediately north of Bal Harbour. The south fill area (2,640,000 cy) covered a 1.5-mile reach immediately south of Bal Harbour between 96th and 80th Streets, covering the entire 1.0-mile length of Surfside, plus a 0.5-mile segment extending into northern Miami Beach. Work began in May 1977 and was completed in September 1978.

The Phase II contract extended the Phase I Surfside/northern Miami Beach fill an additional 1.5 miles southward through Miami Beach, from 80th St southward to 63rd St, with the placement of 1,530,000 cubic yards of material. Construction on Phase II was completed in 1979.

Phase III of the project extended the fill an additional 2.4 miles through Miami Beach, from 63rd Street southward to 36th Street. The total volume of placement was 3,177,100 cubic yards, and construction on Phase III was completed in 1980.

Phase IV of the project extended the fill an additional 1.4 miles to the south, from 36th Street to 17th Street in Miami Beach. The total volume of placement was 2,200,000 cubic yards, and construction on Phase IV was completed in October 1981.

Phase V of the project extended the fill over the final 1.9 miles from 17th Street southward to the southern end of the Federal beach fill project at the Government Cut north jetty. The total volume of placement was 2,400,000 cubic yards, and construction of Phase V was completed in January 1982.

Addition of the Sunny Isles segment to the Dade County BEC & HP project was authorized in 1985, as previously described. This portion of the project consisted of constructing a protective beach fill along the entire 2.4-mile length of Sunny Isles. No protective dune was authorized for this segment of the project. Initial construction of the project was begun in May 1988, and resulted in the placement of 1,320,000 cubic yards of fill along the 2.4-mile length of Sunny Isles.

Maintenance of the Federal Project.

Numerous periodic beach renourishments have been performed under the authority of the BEC & HP project, and several separate placements of beach-quality material dredged from the adjacent Federal navigation projects have occurred along reaches of the project since completion of initial construction. These maintenance-related activities have been performed along most reaches of the Federal project, and selected events are detailed in the following paragraphs in chronological order. Only maintenance events which involved dredging of Bakers Haulover Inlet or fill placement within the limits of Bal Harbour or the adjacent beaches (Haulover Park, Surfside) will be included in this section.

1980 Maintenance Disposal – Haulover Park. Material dredged from the flood shoal of the Federal navigation project at Bakers Haulover Inlet was placed along the Haulover Beach Park shoreline. The volume dredged from the shoal was 43,163 cubic yards.

1984 Maintenance Disposal – Haulover Park. Material was again dredged from the flood shoal at the Federal navigation project at Bakers Haulover Inlet and placed along Haulover Beach Park. The volume dredged from the shoal was 35,000 cubic yards.

1987 Renourishment of Haulover Beach Park. The shoreline within the park was renourished with 235,000 cubic yards of material.

1990 Bal Harbour Renourishment. Approximately 225,000 cubic yards of material were placed along the Bal Harbour shoreline in 1990.

1990 Sunny Isles Renourishment. Approximately 32,000 cubic yards of material removed during the 1990 maintenance dredging of the Federal navigation projects at Bakers Haulover Inlet and the Atlantic Intracoastal Waterway were placed along the northernmost 1,200 feet of Sunny Isles.

1994 Maintenance Disposal – Bakers Haulover Inlet. In February 1994, 24,560 cubic yards of material were dredged from Bakers Haulover Inlet and placed on the adjacent beach at Haulover Park.

1998 Maintenance Disposal – Bal Harbour. A total volume of 282,852 cubic yards of material was removed from the Bakers Haulover Inlet entrance channel, the Intracoastal Waterway, and the IWW approaches to the inlet. This material was placed along a 3,000-foot reach of Bal Harbour, beginning 1,000 feet south of the inlet (extending from DNR survey monument R-28 to R-31). Construction began in May 1998 and was completed in June 1998.

1999 Renourishments of Surfside and South Miami Beach. This contract provided for the placement of 590,000 cubic yards of material along Surfside between 96th and 88th Streets, and 132,000 cubic yards of material along the southernmost 1,500 feet of the project immediately north of the Government Cut north jetty. The Surfside segment was completed in June 1999 and the Miami Beach segment was completed in July 1999.

2003 Bal Harbour Renourishment. Approximately 188,000 cubic yards of fill were placed along the Bal Harbour shoreline between June and August 2003. The fill area extended along the entire 0.85-mile length of Bal Harbour, from the Bakers Haulover Inlet south jetty southward to 96th Street, corresponding to DNR monuments R-27 to R-31.5. A 240-foot wide construction berm was constructed at +9.0 ft mlw elevation, with a front slope of 1v:11h. No work on the project's storm dune was required. The source of beach fill was the ebb shoal at Bakers Haulover Inlet. The ebb shoal borrow area was selectively dredged to varying depths in order to maintain the approximate shape of the shoal. In this manner changes to existing wave refraction patterns over the shoal and to the sheltering effects of the shoal on adjacent shorelines were minimized.

<u>Future Project Modifications.</u>

Two project modifications are currently planned for construction at the time of this writing. Renourishment of northern Miami Beach (R-38 through R-46) is scheduled for 2005, and construction of a breakwater near the southern end of that fill will be constructed in 2005 or early 2006. Due to the distance between these proposed projects and the Bal Harbour area, neither project should have significant effects on the Bal Harbour project.

<u>Previous Studies.</u>

Numerous studies have been performed under the authority of the Dade County BEC & HP project, both for initial project design and to determine the feasibility of subsequent modifications to the project. A chronological listing of major study efforts related to the Bal Harbour segment of the project follows:

<u>General Design Memorandum, September 1975. (reference 1b, in bibliography).</u> This report presented updated detailed design for construction of the Federal project through the communities of Miami Beach, Surfside, and Bal Harbour.

<u>Survey Report and EIS Supplement, June 1982. (reference 1c).</u> This report recommended the addition of the Sunny Isles segment to the Dade County project, and extension of the period of Federal participation in the cost of renourishing the entire project from 10 years to 50 years.

Design Memorandum (CP&E), April 1985. (reference 1d). This report provided a detailed update of the cost of constructing the recommended plan presented in the June 1982 Survey Report, and presented a basis for cost-sharing agreements, preparation of plans and specifications, acquisition of lands, negotiation of relocation agreements, and the scheduling of funding for construction of the project.

General Design Memorandum, Addendum IV, September 1987. (reference 1e). The purpose of this fourth addendum to the 1975 GDM was to examine the performance of the Federal project along the community of Bal Harbour, and to develop an effective plan for renourishment of this area.

Feasibility Report, Coast of Florida Erosion and Storm Effects Study, Region III, October 1996 (reference 1f). This study examined the regional coastal processes along the southeast Florida coast, including Palm Beach, Broward, and Dade counties, and provided recommendations for more effectively managing coastal resources in all three counties.

Evaluation Report, Dade County BEC & HP Project, October 2001 (reference 1g). This study evaluated the performance of the entire Dade County BEC & HP project over the past 20+ years. The report identified several erosional hotspots along the project, including Bal Harbour, and formulated alternatives to reduce the higher erosion rates observed along these areas.

PHYSICAL DATA

General.

The physical data presented in this section consists of water level, wind, and wave measurements and hindcasts, and a discussion of recent storms which have affected the project area. Tidal datums in this report are provided by NOAA, and predicted storm surge elevations are provided by FEMA. The wind and wave data used in the engineering analysis and project design are based on Wave Information Study (WIS) hindcast data produced by the U.S. Army Corps of Engineers' Engineering Research and Development Center (ERDC). These data were used to determine longshore transport rates along the Dade County shoreline, which in turn were used to calculate the updated sediment budget which is presented later in this report.

Much of the data presented in this section was gathered over a considerable length of time; for example the WIS wave record extends across a 40-year period, from 1956 through 1995. Longshore transport rates and sediment budgets have already been calculated in several previous studies using these data, and since these raw data sets have not been appended or superceded, the longshore transport rates and sediment budgets previously calculated will be presented herein and referenced where applicable.

Water Levels.

General. Changes in water levels along the Dade County shoreline occur primarily as a result of three separate processes : astronomical tides, storm surges, and long-term sea level rise. Astronomical tides affect the area on a daily basis, while the effects of significant storm surges are much less frequent. The effects of sea-level rise during the 50-year economic life of the Dade County BEC & HP project are much more gradual and smaller in magnitude than the effects of tides or storm surges.

The effects of water levels on the Federal project are important because in addition to the obvious flooding impacts associated with high water levels, waves will generally break closer to shore causing greater damage, and tidal current velocities through inlets (and the resulting scouring and/or shoaling) are greater when the tidal range is at its highest. The most severe conditions that a beach renourishment project will usually be subjected to will be a combination of astronomical high tide coupled with storm surge, while being subjected to storm wave attack. These conditions occur simultaneously on a fairly frequent basis during the passage of tropical storms and winter 'northeasters'.

Although the most noticeable effects on water levels in the project area are due to astronomical tides and storm surges, the effects of long-term sea level rise cannot be ignored, due to its implications on the long-term management of the project. All three processes which affect ocean water levels will be discussed in the following sections.

Astronomical Tides. Astronomical tides are created by the gravitational pull of the moon and sun, and these tides are predictable in magnitude and timing. The National Oceanic and Atmospheric Administration (NOAA) regularly publishes tide tables for selected locations along the coastlines of the United States and selected locations around the world. These tables provide times of high and low tides, as well as predicted tidal amplitudes.

Tides in the Dade County study area are semidiurnal : two high tides and two low tides during each 24-hour period. Two measures of tidal range are commonly used : mean tidal range and spring tidal range. The mean tide range is defined as the difference between mean high water and mean low water, and represents an average range during the entire monthly lunar cycle. The tidal range is typically greater at any location during periods of a new or full moon, and the spring tide range is the average difference in tidal elevations which occurs semimonthly when the moon is new or full. Both tide ranges are relatively low along the Dade County Atlantic shoreline - the mean tide range is 2.54 feet and the spring tide range is 3.05 feet.

An historical database of astronomical tide data has been compiled by the National Oceanic and Atmospheric Administration (NOAA) at a tide station located at Haulover Beach fishing pier, located near the center of the Haulover Beach Park. This pier was destroyed by Hurricane Andrew in 1992, but tidal benchmarks were established at this station prior to its destruction. These tidal benchmarks are based on 9 years of data, recorded from 1982 to 1990. Table 1 presents the benchmarks computed from this station, referenced to mean low water (mlw). All historical surveys, project design dimensions, prior reports and studies, and construction plans and specifications have been referenced to mlw since the project was authorized and constructed. Although mllw is currently used as the datum for most new projects, the mlw datum will be used in this report to maintain consistency with the historical database.

TABLE 1
NOAA Tidal Datums

Tidal Datum	Elevation, ft above MLW
Highest Observed Water Level (24 Nov 84)	4.56 ft.
Mean Higher high Water (MHHW)	2.60
Mean High Water (MHW)	2.54
Mean Tide Level (MTL)	1.27
Nat'l Geodetic Vert. Datum of 1929 (NGVD29)	0.79
Mean Low Water (MLW)	0.00
Mean Lower Low Water (MLLW)	-0.13
Lowest Observed Water Level (6 June 89)	-1.30

Storm Surges. Storm surge is defined as the rise of the ocean surface above the normal astronomical tide level due to storm effects. Strong onshore winds pile up water near the shoreline, resulting in superelevated water levels along the coastal region and inland waterways. In addition, the lower atmospheric pressure which accompanies storms also contributes to a rise in water surface elevation. Extremely high wind velocities coupled with low barometric pressures (such as those experienced in tropical storms, hurricanes, and very strong northeasters) can produce very high, damaging water levels. For example, a peak surge of up to 10.6 feet mlw was measured along the Key Biscayne shoreline in southern Dade County during the passage of Hurricane Andrew in 1992. Factoring out the 2.5-foot astronomical high tide resulted in a storm surge of 8.1 feet.

Storm surge levels versus frequency of occurrence were calculated for coastal counties throughout Florida by the Federal Emergency Management Agency (FEMA) as part of that agency's Flood Insurance Study (reference 3c). The storm surge elevation vs frequency of occurrence curves for Dade County are shown in figure 5 below.

Methodology developed by the National Academy of Sciences was used in the development of these series of storm surge curves. These curves do not include tidal effects, but do include the effects of storm waves, and also include the wave dissipation effects of features such as dunes and vegetation, and coastal structures such as seawalls and buildings. Two sets of curves are provided on the graph in figure 5, consisting of a pair of hurricane-produced surge levels, and a pair of northeaster-produced surge levels.

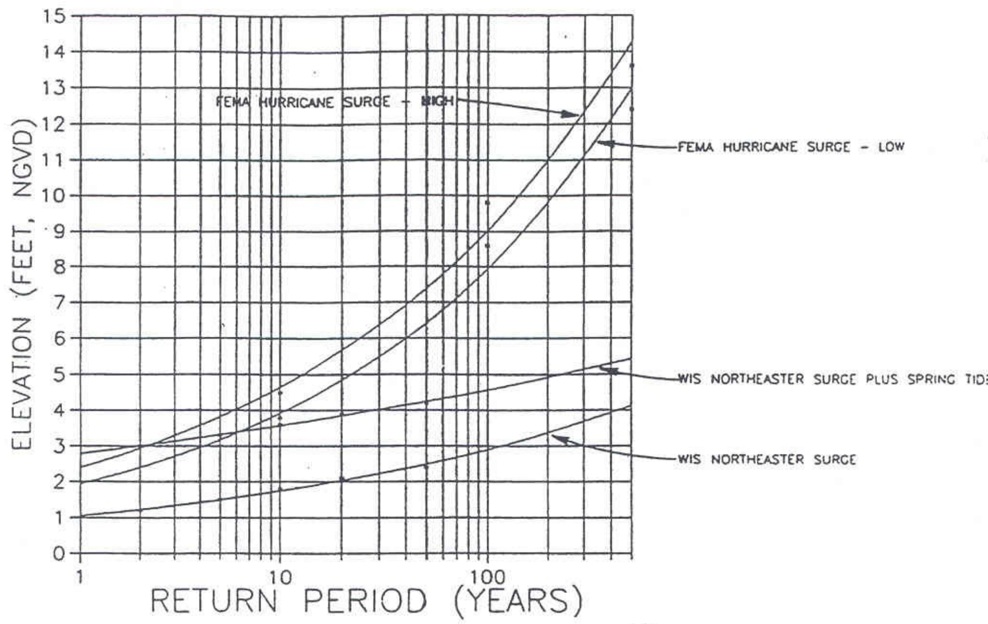


Figure 5. FEMA Storm Surge Frequency Curve.

The FEMA hurricane surge curves are interpolated from data points for the 10, 50, 100, and 500-year recurrence hurricane events. The 'high' curve was computed for hurricane surges along the southern portion of Dade County, while the 'low' curve was calculated for surges along the northern portion of the county. The differences between these two curves are mainly due to differences in wave runup levels at the northern and southern ends of the county.

Storm surge levels due to northeasters were generated from WIS data hindcasts. The 'northeaster' curves shown on this graph were interpolated from data points for the 2, 5, 10, 20, and 50-year recurrence events at Miami Beach. Like the FEMA hurricane data, the WIS northeaster data does not include astronomical tide, so two 'northeaster' surge curves are provided in figure 5 : the lower curve shows surge only, while the upper curve shows the spring high tide superimposed on the lower curve. Since most northeasters last several days, the upper northeaster curve represents a realistic 'worse-case' scenario. The FEMA hurricane surge curves are extrapolated below the 10-year event, and the WIS northeaster curves are extrapolated above the 50-year event, so care should be used when determining surge levels based on these extrapolated portions of the storm surge curves.

Sea Level Rise. Eustatic sea level change is defined as a global change in the water surface elevations of the world's oceans. The total relative sea level change is the difference between the eustatic sea level and changes in local land surface elevations, and may include the rise and fall of the land as well as changes of the eustatic sea level. The eustatic sea level has varied widely over geologic time, and evidence suggests that sea levels in the past have been much higher, and much lower, than present-day levels.

Currently there is considerable debate about whether sea levels may be dropping due to the onset of a new ice age, or whether sea levels are rising due to polar ice cap melting associated with global warming. Research on this subject has been divided as to whether the long-term outlook favors a eustatic sea level rise or fall. Four widely-circulated studies have been performed since the Environmental Protection Agency first addressed the issue of sea-level rise in 1983. Figure 6 shows a graphic representation of the results of each study, including the year in which it was published, the predicted amount of sea level rise or fall by the year 2100, and the degree of uncertainty. As seen in this figure, the original 1983 study indicated the greatest sea level rise, at 2.0 meters by the year 2100. Subsequent studies by the National Research Council (NRC) have resulted in lower predictions, with smaller ranges of uncertainty. The latest NRC study was performed in 1990, and predicts a sea level rise of 0.5 meters by 2100, with a wider range of uncertainty : plus or minus 1 meter. According to long-term gage measurements recorded at Miami Beach by NOAA, the measured sea level rise from 1931 to 1981 is 2.39 mm per year (0.78 ft/century).

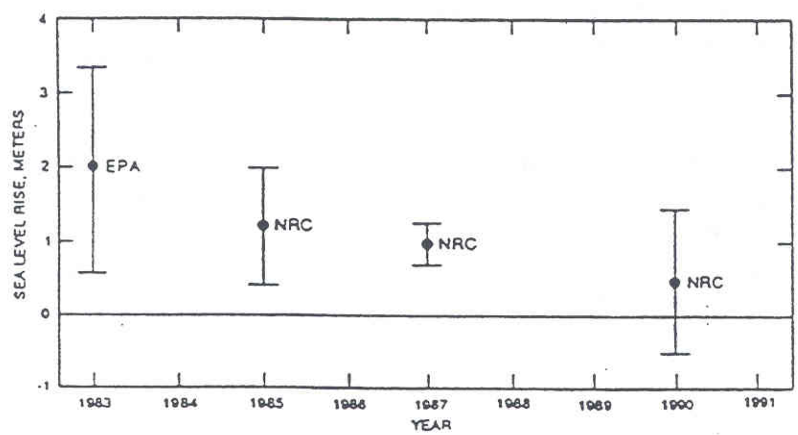


Figure 6. Summary of Predicted Sea Level Rise by Year 2100

(Houston, 1993)

The 1987 report published by the NRC recommended that sea level rise should be considered during the initial design of coastal projects. The report did not suggest specific new analysis techniques or guidelines. However, most Corps of Engineers shore protection projects, including the Bal Harbour segment of the Dade County BEC & HP project, have a 50-year economic life, and renourishment of the project will typically be required several times during this 50-year period. The feasibility of future renourishments can be reevaluated based on any significant changes to project conditions, such as sea level rise. Monitoring of eustatic sea levels over the past few decades indicates that any such large-scale sea level changes during this period have occurred very slowly, and at this point have had no significant impact on the overall management of shore protection projects such as the Dade County BEC & HP over the 50-year economic life of the project. However, a re-evaluation of the project should be conducted if significant changes in eustatic sea levels occur.

Wind Data.

Local winds in the project area are the primary means of generating the small-amplitude, short period waves which impact the south Florida shoreline for much of the year. Dade County lies at 26 degrees latitude, within the edges of the tropical tradewind zone. Winds in this zone originate from the northeast, east, and southeast much of the year, with the greatest velocities originating from the northeast (in winter months), and the greatest frequencies of occurrence from the east and southeast (spring, summer, early fall).

Table 2 shows a summary of wind data, from WIS Station 9, located at latitude 26.0 degrees north, longitude 80.0 degrees west, or approximately 6 nautical miles directly offshore from the Dade/Broward county line. This table contains monthly summaries of windspeed and direction which illustrate the strong seasonal variation described above.

TABLE 2
Wind Data Summary - Offshore Dade County

Windspeed m/sec (mph)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total	Percent Occurrence
	Wind Speeds - WIS Revised Phase II, Station 9													
0.00-2.49 (0.0-5.5)	202	177	206	249	465	553	550	519	420	246	167	158	3912	6.7
2.50-4.99 (5.6-11.1)	2202	1993	2199	2302	2849	3354	3650	3613	2987	2433	1973	1922	31477	53.9
5.00-7.49 (11.2-16.7)	1295	1156	1542	1389	1228	597	681	766	1051	1155	1246	1460	13566	23.2
7.50-9.99 (16.8-22.2)	908	865	823	755	408	245	79	60	307	794	957	1081	7282	12.5
10.00-12.49 (22.3-27.8)	262	260	153	85	9	34	0	2	28	262	347	276	1718	2.9
12.50-14.99 (27.9-33.4)	85	68	37	15	1	17	0	0	7	68	104	61	463	0.8
15.00-17.49 (33.5-39.0)	5	1	0	5	0	0	0	0	0	2	6	2	21	0.0
17.50-19.99 (39.1-44.5)	1	0	0	0	0	0	0	0	0	0	0	0	1	0.0
20.00+ (44.6+)	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
Total	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440	100.0

Center of 45-deg band	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total	Percent Occurrence
	Wind Direction - WIS Revised Phase II, Station 9													
0.00	650	565	492	402	259	117	61	158	286	653	758	727	5128	8.8
45.0	1194	740	730	798	755	510	371	694	1192	1836	1529	1415	11764	20.1
90.0	1107	903	1061	1481	1797	1556	2295	2049	1668	1223	1224	1237	17601	30.1
135.0	658	676	909	781	816	1097	1102	1006	772	430	339	525	9111	15.6
180.0	359	479	547	421	482	633	521	418	301	159	150	237	4707	8.1
225.0	251	305	330	258	291	436	313	293	244	139	164	164	3188	5.5
270.0	282	330	364	358	312	298	204	194	176	199	213	205	3135	5.4
315.0	459	522	527	301	248	153	93	148	161	321	423	450	3806	6.5
Total	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440	100.0

From February through September, winds originate most frequently from the eastern 45-degree sector, with a stronger tendency toward the southeasterly directions from March through August. From October through January winds originate more from the northeast sector as cold fronts move through the region, usually associated with 'northeasters' of varying intensities.

A summary of wind directions at the right side of the bottom of table 2 shows that winds originating from the east and southeast sectors occur 30 and 16 percent of the time, respectively, while winds from the northeast sector occur 20 percent of the time. An analysis of the top portion of the table shows that wind velocities are generally higher in the late fall and winter months when northeasters generate strong winds from the northeast sector which can last for several days per storm event. The overall wind climate can be classified as moderate- to low-energy, with windspeeds less than 22 mph occurring 96 percent of the time, and windspeeds less than 16 mph 84 percent of the time.

Wave Data.

General. The project area is vulnerable to wave attack from locally-generated waves from all of the easterly directions. The overall wave climate is moderate and as a result, locally-produced waves are generally small and low-energy relative to most of the east coast of the U.S. The study area is shielded from all but the most northerly open-ocean storm swells by the Bahama Bank, so with a few exceptions distant storm swells do not cause significant damage within the project area. The presence of the Bahama Bank 60 miles offshore also limits the generation of fetch-limited waves.

The majority of damages to the Federal project and to upland development along Dade County's shorelines are caused by wave attack. Two primary modes of wave-induced damages are experienced along the Dade County shoreline : large-scale episodic damages to the project and upland development result from storm wave impacts from the relatively infrequent passage of hurricanes, tropical storms, and strong northeasters; and the slower but more persistent erosional damage to the Federal project which results from the nearly constant attack of smaller locally-generated wind waves on the county's shoreline. Both types of wave attack have played a part in the historical erosional damages to Bal Harbour, and each will be examined in greater detail in this section.

The largest waves which impact the Dade County shoreline are produced by nearby tropical disturbances, including hurricanes, and those northeasters which produce strong swells from the most northerly directions. The extreme southerly location of the project area on the Florida peninsula usually results in weaker local effects from the passage of northeasters, since these large-scale weather systems tend to lose strength as they move further south into warmer climates and over the warmer waters of the lower latitudes. The reverse is true for hurricanes and other tropical disturbances, and as a result Dade County lies near the center of a broad corridor of hurricane activity. Figure 7 is a plot produced by NOAA which shows tracks of hurricanes that have passed within 50 nautical miles of the Dade County shoreline between 1871 and 1993. Records indicate that 29 hurricanes of at least Category 1 intensity (winds over 74 mph) have passed within 50 nm of the project area over this 124-year period, equating to one occurrence of a Category 1 (or higher) storm every 4.3 years.

Deepwater Wave Conditions. The most detailed long-term database available for this region is the revised Wave Information Study (WIS) phase II wave hindcast produced by ERDC for the Atlantic coast of the U.S. This WIS data was used in this study to define wave characteristics of the area, and was used as an input database for numerical shoreline modeling. The original WIS record extends from 1956 through 1975, and was later appended with a data set which extends from 1976 through 1995. The latter 20-year data set includes the effects of tropical disturbances; the earlier 20-year set does not. Both data sets consist of a time-series listing of wave height, period and directions at 3-hour intervals throughout the hindcast period. The locations of the revised WIS phase II stations along the southeast Florida coast are shown in figure 8. Station 9 is located closest to the project area, 6 nautical miles offshore of the Dade/Broward county line, at latitude 26.0 deg N, longitude 80.0 deg W. The station is in 220 meters (720 feet) of water, so virtually the entire WIS record can be considered to be deep-water waves at the station location.

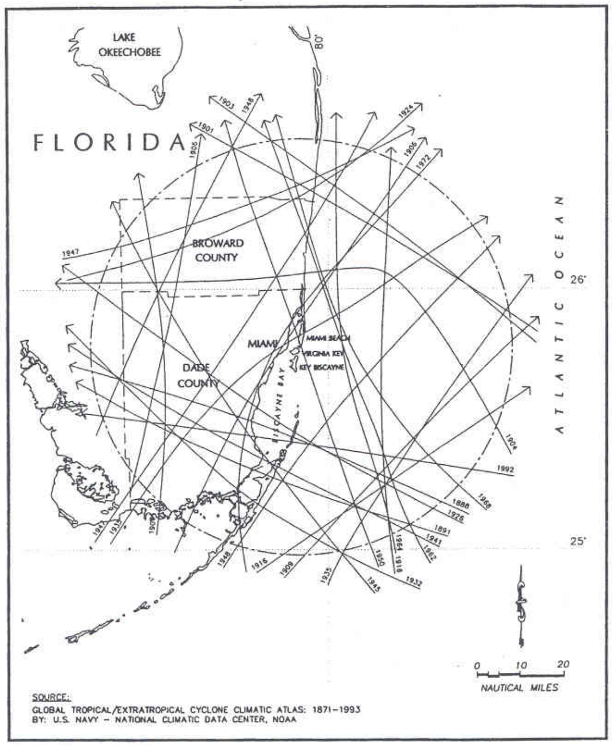


Figure 7. NOAA Hurricane Tracks Within 50 nm of Dade County (1871 – 1993)

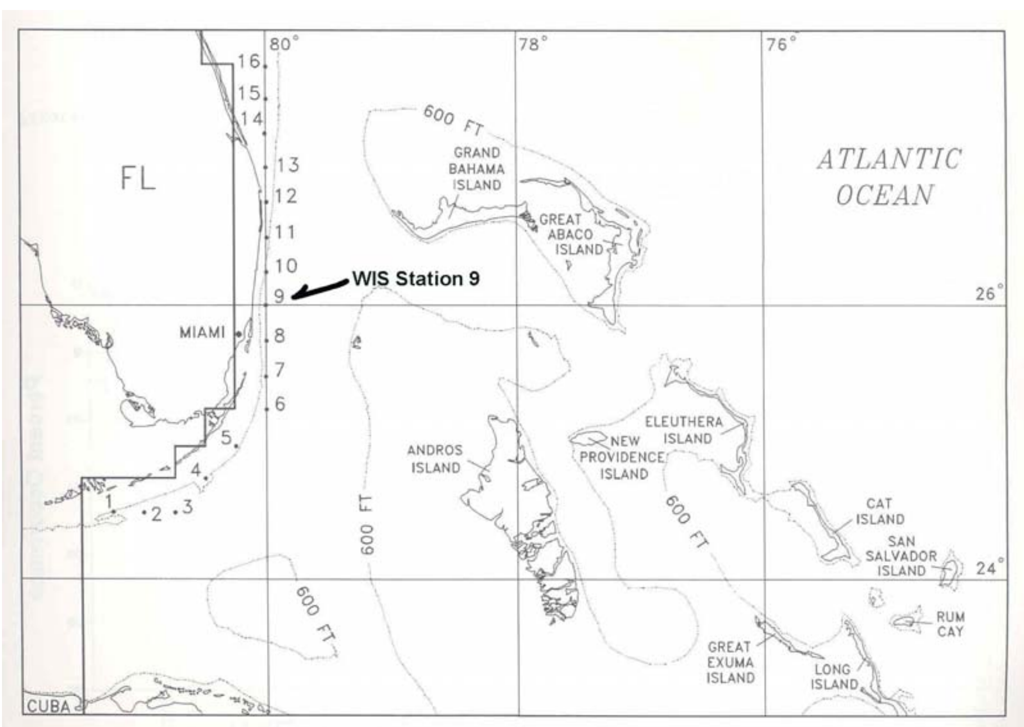


Figure 8. WIS Station 9 Location Map

This revised WIS database supercedes the original WIS phase II database by using the newer WISWAVE 2.0 numerical model to generate wave data from windfields. WISWAVE 2.0 allows the use of a finer nearshore grid, and includes more bathymetric data from the nearshore region than the original WIS phase II hindcast. WISWAVE 2.0 also includes the capability to model the sheltering effects of the Bahamas Bank, which lies directly offshore of the southeast Florida coastline. This improvement is considered essential for providing realistic wave input data for numerical shoreline simulation models, as the presence of the Bahama Bank (see figure 8) prevents long-period ocean swells from reaching the Dade County shoreline from all but the most northerly directions, and limits the generation of wind waves under a wide variety of conditions.

Wave generation closely parallels the wind speed and directions discussed in the previous section, as shown in the summary of 20 years of WIS wave data at station 9 provided in table 3. These summary tables were generated from the time-series of hindcast data from the period 1976-95. Table 3 consists of four separate tables which summarize occurrences of significant wave height (table 3a), peak wave period (table 3b), peak wave direction (table 3c), and yearly/monthly mean wave heights (table 3d) at the deepwater location of WIS station 9. All four summary tables categorize data by month, so that seasonal variations in wave characteristics can be easily observed.

The bathymetry offshore of southeast Florida is somewhat unusual for the east coast of the U.S., in that the continental shelf is at its narrowest along Palm Beach, Broward, and Dade Counties. The typical width of the continental shelf along the Dade County shoreline is about 2 miles. Due to the large degree of transformation which occurs as deepwater waves propagate from the WIS station location in 720 feet of water to breaking depths along the shoreline, two sets of wave data will be discussed. The deepwater wave database summarized in table 3 was transformed to the 12-foot depth contour using a Snells Law routine, and the resulting shallow-water database is summarized in table 4. Deepwater data summarized in table 3 will be analyzed first, then the transformed wave data contained in table 4 will be discussed.

Analyses of hindcast wave data and field observations of the wave environment along the southeast Florida coast indicate that strong seasonal effects exist, similar to the wind data tendencies. Waves during the summer months rarely exceed 1-2 feet, and usually originate from the east/southeasterly directions. These locally-generated wind waves occur the vast majority of the time throughout the year, with the exception of a shift toward more frequent occurrences of larger storm-generated waves from the northerly directions during the winter months. Historically, calculated net littoral drift rates along the southeast Florida coast have indicated a net transport direction to the south, with a large northerly transport component during the summer months.

STATION: 9

Table 3a - OCCURRENCES OF WAVE HEIGHT BY MONTH FOR ALL YEARS

Hmo(ft)	JAN	FEB	MAR	APR	MAY	JUNE	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	PERCENT
0.00 - 1.99	1038	868	1207	1346	2038	3105	3654	3342	2250	1423	814	1041	22126	37.86
2.00 - 3.99	2308	2139	2010	2318	2110	1363	1167	1301	1952	1908	2079	2186	22841	39.08
4.00 - 5.99	1015	945	968	733	653	261	106	208	404	1087	1175	1008	8563	14.65
6.00 - 7.99	371	345	474	260	120	50	23	69	96	389	458	425	3080	5.27
8.00 - 9.99	114	128	203	94	29	19	9	14	56	112	168	206	1152	1.97
10.00 - 11.99	59	61	63	23	9	2	1	5	31	37	57	54	402	0.69
12.00 - 13.99	26	26	25	26	1	0	0	6	2	4	30	24	170	0.29
14.00 - 15.99	13	6	6	0	0	0	0	12	3	0	9	12	61	0.10
16.00 - 17.99	9	2	3	0	0	0	0	1	3	0	5	4	27	0.05
18.00 - 19.99	5	0	1	0	0	0	0	0	2	0	3	0	11	0.02
20.00 - Greater	2	0	0	0	0	0	0	2	1	0	2	0	7	0.01
Total	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440	100.00

Table 3b - OCCURRENCES OF PEAK PERIOD BY MONTH FOR ALL YEARS

Tp(sec)	JAN	FEB	MAR	APR	MAY	JUNE	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	PERCENT
3.00 - 3.9	161	134	209	195	368	656	938	892	354	90	119	118	4234	7.25
4.00 - 4.9	694	556	601	600	807	894	1423	1052	556	384	449	616	8632	14.77
5.00 - 5.9	861	832	759	915	922	690	710	663	650	652	894	732	9280	15.88
6.00 - 6.9	820	758	772	726	763	826	533	530	607	855	981	842	9013	15.42
7.00 - 7.9	478	497	566	399	436	443	390	438	492	648	620	550	5957	10.19
8.00 - 8.9	327	332	446	318	331	326	293	400	484	473	378	441	4549	7.78
9.00 - 9.9	285	269	244	201	303	222	173	194	443	468	290	316	3408	5.83
10.00 - 10.9	189	228	217	173	200	201	130	162	281	292	222	203	2498	4.27
11.00 - 11.9	178	112	205	213	212	221	151	147	207	224	164	151	2185	3.74
12.00 - 12.9	209	99	189	315	301	135	131	198	216	216	178	176	2363	4.04
13.00 - 13.9	147	126	194	322	147	88	54	136	196	205	130	216	1961	3.36
14.00 - 14.9	137	114	223	192	76	55	17	61	137	193	150	191	1546	2.65
15.00 - 15.9	120	91	119	89	60	28	9	37	113	112	52	164	994	1.70
16.00 - 16.9	149	144	67	68	13	10	6	21	34	66	80	80	738	1.26
17.00 - 17.9	99	117	49	20	12	5	2	20	16	39	38	55	472	0.81
18.00 - 18.9	65	56	45	16	4	0	0	8	11	21	22	40	288	0.49
19.00 - 19.9	27	22	36	27	3	0	0	1	3	11	20	25	175	0.30
20.00 - Longer	14	33	19	11	2	0	0	0	0	11	13	44	147	0.25
Total	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440	100.00

Table 3c - OCCURRENCES OF PEAK DIRECTION BY MONTH FOR ALL YEARS

Dp(deg)	JAN	FEB	MAR	APR	MAY	JUNE	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	PERCENT
348.75 - 11.24	535	347	272	197	74	45	11	45	52	115	289	394	2376	4.07
11.25 - 33.74	980	844	548	493	474	369	184	408	787	1064	974	1166	8291	14.19
33.75 - 56.24	1495	1255	1635	1738	1556	1199	828	1265	1973	2384	1541	1609	18478	31.62
56.25 - 78.74	311	355	377	365	488	400	257	383	573	645	631	400	5185	8.87
78.75 - 101.24	396	449	512	686	870	690	1156	1093	636	357	676	474	7995	13.68
101.25 - 123.74	225	307	369	375	573	726	1062	682	315	126	265	280	5305	9.08
123.75 - 146.24	218	243	384	408	459	620	1015	636	231	98	172	161	4645	7.95
146.25 - 168.74	205	198	301	194	247	437	316	281	127	63	69	112	2550	4.36
168.75 - 191.24	139	131	172	60	107	196	63	96	80	42	49	64	1199	2.05
191.25 - 213.74	43	34	27	16	16	46	34	21	10	9	11	25	292	0.50
213.75 - 236.24	26	11	35	9	8	18	19	28	5	8	13	17	197	0.34
236.25 - 258.74	19	18	28	20	13	34	12	3	2	4	5	10	168	0.29
258.75 - 281.24	20	31	52	27	33	15	2	4	0	5	11	16	216	0.37
281.25 - 303.74	83	60	76	52	21	2	1	2	0	4	7	23	331	0.57
303.75 - 326.24	101	113	81	81	12	1	0	5	0	10	27	92	523	0.89
326.25 - 348.74	164	124	91	79	9	2	0	8	9	26	60	117	689	1.18
Total	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440	100.00

Table 3d - SUMMARY OF MEAN Hmo (ft) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUNE	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1976	4.27	3.54	3.31	3.13	2.79	2.19	1.39	2.16	1.36	4.42	4.26	5.03	3.15
1977	3.76	3.34	3.62	4.93	3.12	1.36	1.73	3.07	2.02	3.01	4.53	3.96	3.21
1978	4.00	4.10	3.42	2.95	2.24	2.10	1.90	1.81	2.51	4.66	4.11	4.63	3.20
1979	5.59	4.20	4.41	4.42	3.07	2.57	2.09	1.59	4.39	2.96	5.21	4.40	3.74
1980	3.45	4.73	3.79	2.99	2.43	1.79	1.39	2.60	1.97	2.62	4.38	4.37	3.04
1981	3.37	5.57	4.68	3.57	2.28	2.18	1.54	2.67	2.59	3.47	3.82	3.25	3.25
1982	3.15	2.88	3.41	2.56	2.81	2.37	1.50	1.69	2.03	3.55	3.87	4.17	2.83
1983	2.98	4.63	3.84	3.63	2.90	1.88	1.74	1.54	3.06	3.68	3.23	4.49	3.13
1984	5.28	3.89	3.85	3.05	3.46	2.30	1.97	1.67	3.85	4.84	5.64	4.25	3.67
1985	3.21	4.58	3.53	3.49	1.80	1.70	1.75	2.25	4.36	3.37	4.37	4.11	3.21
1986	4.00	3.20	4.92	2.82	3.55	1.88	1.45	2.46	2.91	3.45	3.80	4.33	3.23
1987	3.97	3.63	5.48	2.88	2.86	2.14	2.08	1.80	1.55	4.28	4.49	3.21	3.20
1988	4.65	3.48	3.41	2.92	2.73	2.52	1.90	1.78	3.13	3.24	3.07	2.96	2.98
1989	2.95	3.18	3.41	2.44	2.07	1.81	1.48	1.46	2.46	3.04	2.44	2.78	2.46
1990	2.91	4.32	4.11	3.40	2.81	2.11	1.97	1.30	2.20	3.15	3.76	3.69	2.98
1991	2.88	3.20	3.35	3.29	3.09	1.82	1.32	1.57	1.97	3.50	3.73	3.37	2.76
1992	3.17	3.07	2.84	3.34	2.64	1.99	2.12	2.12	2.32	3.34	4.46	3.51	2.91
1993	4.30	3.75	4.03	3.58	3.15	2.40	1.35	1.79	2.37	2.57	4.09	3.55	3.08
1994	4.54	3.92	3.37	3.49	2.58	1.90	2.35	2.15	2.57	3.02	4.36	4.09	3.20
1995	3.34	3.08	4.24	2.98	2.43	2.44	2.07	2.99	2.76	4.13	3.75	3.60	3.15
MEAN	3.79	3.82	3.85	3.29	2.74	2.07	1.75	2.02	2.62	3.52	4.07	3.89	

Table 3 – WIS deepwater summary tables.

This seasonal effect is reflected in the data contained in tables 3 and 4. Table 3a provides monthly summaries of deepwater significant wave heights for the 20-year period from 1976-95, and demonstrates the seasonal variation of deepwater wave heights between the summer and winter months. During the summer months wave heights are generally much lower than in the winter months, and are concentrated in the lowest wave-height band in this table. During the months of August and September the overall wave energy is still low, but several large storm events are observed; these are the products of tropical disturbances and early 'northeasters'. During the winter months (November to March) the overall wave energy is somewhat higher, with the majority of wave events in each month falling in the 2.0-3.99 ft range, and a greater number of large storm waves produced by winter 'northeaster' storms.

On an annual basis, 77 percent of all deepwater wave events fall within the lower 2 wave height bands (0 – 3.99 feet). Of the larger wave events in the 20-year record, only 1.2 percent exceed 10 feet, and many of these events occur in August and September and are the results of tropical disturbances. An analysis of table 3a indicates that on average, the southeast Florida coast experiences a relatively low-energy wave environment year-round, but a pronounced increase in overall wave energy occurs between the summer and winter months.

Wave periods show the same seasonality as wave heights; short-period, locally-generated wind waves are common throughout the year, but in the summer months these short period waves occur almost exclusively. During the winter months a shifting towards higher-energy, longer-period storm swells can be seen in the monthly summary of occurrences of peak wave periods in table 3b. From the summary columns in table 3b, it is seen that 63.5 percent of all deepwater waves fall in the peak period bands of 0-7.9 seconds, a category which is widely accepted as locally-generated 'wind waves'. Of the remaining 36.5 percent of the database, 31.7 percent of the wave periods fall within the 8.0-14.9 second band. Storm-produced swells with periods longer than 15 seconds occur in only 4.8 percent of all wave events and given their occurrences in the months of October through May, the majority of these events were very likely generated by strong northeasters.

Table 3c contains a summary of occurrences of peak wave direction during the 20-year WIS hindcast record, with the direction of wave incidence broken into 22.5-degree bands. Again seasonal effects are noted. An examination of table 3c shows that waves originating from the east and northeast are prevalent throughout much of the year, but during July and August peak directions shift to more southeasterly directions. Again, the presence of the Bahama Bank limits the available fetch for the growth of local wind-generated waves, and prevents distant ocean swells from the easterly directions from reaching the location of WIS station 9 and the Dade County shoreline.

Table 3d provides a summary of mean wave height by month and year throughout the 20 year period of this analysis. The difference in relative wave energy between summer and winter months is easily seen in this table, both for individual years and in the summary of mean wave heights presented in the bottom row of the table. By comparing table 3a with table 3d, it is seen that the higher mean wave heights during the winter months are the result of a relatively few large storm events, and deepwater wave heights still remain below 4 feet the majority of the time. These tables again demonstrate that the overall wave energy at the deepwater location of WIS station 9 is relatively low, especially during the summer months.

Shallow-Water Wave Conditions. The wave environment differs considerably between the deepwater WIS station location and the nearshore region in the vicinity of Bal Harbour which is of interest in this report. Table 4 consists of a series of four tables similar to those contained in table 3, except that the wave events represented in this table have been transformed to near-breaking depths along the 12-foot depth contour along southern Bal Harbour, which is located about 300 to 400 feet offshore. The wave events depicted in table 4 were transformed using Snell's Law, which neglects any local effects of wave focusing caused by irregularities in the offshore bathymetry. The bathymetry is highly irregular offshore of Bal Harbour however, due to the presence of the Bakers Haulover Inlet ebb shoal along the northern portion of the project area, and due to the presence of multiple rows of shore-parallel coral reefs throughout the region. The purpose of this analysis is only to provide an indication of the differences in deepwater versus shallow-water wave field characteristics. The effects of bathymetry and wave-current interaction are very complex, and will be the subject of detailed analysis in the Numerical Modeling section of this report.

The offshore location of WIS station 9 is subjected to waves originating from all directions, but the nearshore location of the data contained in table 4 is subjected only to waves originating from the easterly directions due to the sheltering effect of the shoreline. Events recorded in table 3 as originating from the westerly directions are therefore recorded in table 4 as "calm" events, with a wave height of 0 feet. These calm events are excluded from the period and direction summary tables (tables 4b and 4c, respectively).

A shortcoming of Snell's Law is that no allowance is made for breaking waves. All waves higher than about 9.4 feet will break before reaching the 12-foot contour, but it is seen that many wave events much higher than 9.4 feet are recorded at the 12-foot contour in table 4a. Between 2,854 and 5,662 wave events will exceed 9.4 feet in height at the 12-foot contour, depending on how many of the events in the 8 to 9.99-foot wave height band are over 9.4 feet. This constitutes from 4.9 to 9.7 percent of the total WIS wave record of 58,440 events. However, since the 12-foot depth contour lies approximately 300 to 400 feet offshore, it can be safely assumed that the waves which break seaward of the 12-foot contour will have the same (or less) impact on nearshore sediment transport processes than a 9.4-foot wave.

A comparison of data between tables 3a and 4a shows that wave amplitudes are slightly higher at the 12-foot contour due to shoaling, but the overall seasonal effects remain unchanged. Approximately 6.4 percent of the wave record consists of the "calms" at the nearshore station which equate to the westerly waves at the offshore location of station 9. A total of 3735 westerly events occur in the WIS time series (which consists of 58,440 wave events); all of these calms were placed in the 0 – 1.99-ft wave height band in table 4a, in the months in which they occurred.

WIS Station 9 Data

12ft WATER DEPTH

Table 4a - OCCURRENCES OF WAVE HEIGHT BY MONTH FOR ALL YEARS

Hmo(ft)	JAN	FEB	MAR	APR	MAY	JUNE	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	PERCENT
0.00 - 1.99	1056	920	1086	922	1377	2389	3251	2820	1296	703	618	818	17256	29.53
2.00 - 3.99	1502	1383	1418	1749	2058	1798	1425	1500	2234	1766	1385	1518	19736	33.77
4.00 - 5.99	1155	1100	1093	1275	1015	440	223	429	866	1265	1327	1257	11445	19.58
6.00 - 7.99	553	450	460	412	308	108	31	101	185	572	649	512	4341	7.43
8.00 - 9.99	349	315	392	242	123	37	17	57	90	387	421	378	2808	4.80
10.00 - 11.99	141	147	238	90	53	15	8	20	41	140	181	235	1309	2.24
12.00 - 13.99	85	102	142	60	17	11	4	8	41	77	105	134	786	1.34
14.00 - 15.99	36	40	56	14	5	2	0	0	21	31	43	40	288	0.49
16.00 - 17.99	29	26	38	13	3	0	1	4	15	14	24	28	195	0.33
18.00 - 19.99	12	20	21	18	1	0	0	3	1	5	13	18	112	0.19
20.00 - 21.99	19	11	11	5	0	0	0	9	3	0	17	11	86	0.15
22.00 - 23.99	4	4	4	0	0	0	0	6	1	0	6	6	31	0.05
24.00 - 25.99	4	2	0	0	0	0	0	1	3	0	2	2	14	0.02
26.00 - Greater	15	0	1	0	0	0	0	2	3	0	9	3	33	0.06
Total	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440	100.00

Table 4b - OCCURRENCES OF PEAK PERIOD BY MONTH FOR ALL YEARS

Tp(sec)	JAN	FEB	MAR	APR	MAY	JUNE	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	PERCENT
3.00 - 3.9	61	74	106	99	273	549	871	814	333	73	78	59	3390	6.20
4.00 - 4.9	352	291	343	395	755	847	1399	1037	535	329	356	361	7000	12.80
5.00 - 5.9	682	685	644	867	905	672	708	656	632	635	831	637	8554	15.64
6.00 - 6.9	747	692	723	704	756	812	533	519	603	842	935	791	8657	15.82
7.00 - 7.9	444	478	547	392	432	441	390	433	485	635	611	550	5838	10.67
8.00 - 8.9	318	327	437	315	331	326	293	400	482	470	373	441	4513	8.25
9.00 - 9.9	280	269	231	201	303	222	173	194	443	467	288	316	3387	6.19
10.00 - 10.9	189	228	217	173	200	201	130	162	281	292	222	203	2498	4.57
11.00 - 11.9	178	112	205	213	212	221	151	147	207	224	164	151	2185	3.99
12.00 - 12.9	209	99	189	315	301	135	131	198	216	216	178	176	2363	4.32
13.00 - 13.9	147	126	194	322	147	88	54	136	196	205	130	216	1961	3.58
14.00 - 14.9	137	114	223	192	76	55	17	61	137	193	150	191	1546	2.83
15.00 - 15.9	120	91	119	89	60	28	9	37	113	112	52	164	994	1.82
16.00 - 16.9	148	144	67	68	13	10	6	21	34	66	80	80	737	1.35
17.00 - 17.9	99	117	49	20	12	5	2	20	16	39	38	55	472	0.86
18.00 - 18.9	65	56	45	16	4	0	0	8	11	21	22	40	288	0.53
19.00 - 19.9	27	22	36	27	3	0	0	1	3	11	20	25	175	0.32
20.00 - Longer	14	33	19	11	2	0	0	0	0	11	13	44	147	0.25
Total	4217	3958	4394	4419	4785	4612	4867	4844	4727	4841	4541	4500	54705	100.00

Table 4c - OCCURRENCES OF PEAK DIRECTION BY MONTH FOR ALL YEARS

Dp(deg)	JAN	FEB	MAR	APR	MAY	JUNE	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	PERCENT
348.75 - 11.24	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
11.25 - 33.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
33.75 - 56.24	28	6	12	5	17	1	17	20	3	11	23	14	157	0.29
56.25 - 78.74	1286	1075	841	782	827	705	483	782	1313	1712	1514	1530	12850	23.49
78.75 - 101.24	2405	2330	2768	2948	3065	2426	2771	2863	2965	2907	2714	2623	32785	59.93
101.25 - 123.74	495	538	753	681	837	1397	1485	1077	431	211	283	327	8515	15.57
123.75 - 146.24	3	9	20	3	39	83	111	102	15	0	7	6	398	0.73
146.25 - 168.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00
Total	4217	3958	4394	4419	4785	4612	4867	4844	4727	4841	4541	4500	54705	100.00

Table 4d - SUMMARY OF MEAN Hmo (ft) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUNE	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1976	5.17	4.27	4.12	4.35	3.25	2.61	1.63	2.84	1.98	6.42	5.53	6.24	4.03
1977	5.27	3.64	4.85	6.53	3.80	1.50	1.95	3.72	2.58	3.75	6.15	5.40	4.10
1978	3.55	6.03	4.27	3.51	2.60	2.65	2.30	2.12	3.41	6.50	5.33	6.25	4.04
1979	7.48	5.33	6.04	6.20	3.96	3.24	2.53	1.83	5.67	4.05	6.88	6.08	4.94
1980	4.69	6.79	5.35	4.14	3.27	2.24	1.57	3.67	2.52	3.68	6.70	6.16	4.23
1981	3.73	7.76	7.39	4.65	2.97	2.80	1.83	2.97	3.61	4.61	5.33	3.68	4.28
1982	4.53	3.66	4.56	2.86	3.63	2.89	1.65	1.82	2.48	5.15	5.23	5.25	3.64
1983	3.18	6.32	3.50	4.01	3.48	2.23	1.99	1.67	4.11	5.45	3.97	5.64	3.79
1984	7.50	4.52	4.66	4.21	4.20	2.46	2.13	1.80	5.13	6.59	7.77	5.41	4.70
1985	3.53	6.36	4.48	5.08	2.07	1.92	2.06	2.66	5.71	4.32	6.14	5.64	4.16
1986	4.79	4.00	6.06	3.74	5.08	2.18	1.54	2.93	3.75	4.82	5.16	6.01	4.17
1987	5.22	4.27	8.06	3.24	3.49	2.46	2.48	2.30	2.24	5.43	5.55	3.91	4.05
1988	6.14	3.84	3.76	3.93	3.57	3.07	2.18	2.22	3.82	4.44	3.41	3.96	3.70
1989	3.87	3.49	4.27	3.25	2.48	2.07	1.60	1.86	3.59	4.09	2.87	3.71	3.09
1990	3.37	5.25	5.03	4.31	3.52	2.58	2.27	1.78	2.98	3.76	4.51	4.62	3.66
1991	3.14	3.03	3.43	3.99	3.74	2.48	1.38	1.91	2.71	4.68	4.52	4.22	3.27
1992	2.96	3.18	2.82	3.52	3.45	2.28	2.49	2.55	2.90	4.13	5.45	4.42	3.35
1993	5.61	4.12	5.43	4.29	4.00	2.85	1.48	2.10	3.07	3.07	5.18	4.10	3.77
1994	4.94	4.77	3.62	4.19	3.24	2.01	2.53	2.46	3.19	3.91	5.81	5.80	3.87
1995	3.12	2.86	5.02	3.62	3.15	2.61	2.12	3.91	3.64	5.21	3.88	3.47	3.55
MEAN	4.59	4.67	4.84	4.18	3.45	2.46	1.98	2.45	3.45	4.70	5.27	5.00	

Table 4 a-d. Shallow-water wave summary

An examination of table 4 shows many similarities in the distribution of wave heights, periods, and directions as was seen in the summary of deepwater wave data presented in table 3. Wave height distributions in table 4a show the same seasonal effects as were observed in table 3a. Similar to table 3a, waves in the lowest height band dominate during the summer months (June through August), and waves in the 2 to 3.99-foot band dominate during the remainder of the year. However, a large percentage of the total number of wave events in each month falls within the lower height bands at both the deep- and shallow-water locations. Waves less than 2 feet in height at the 12-foot contour occur 29.5 percent of the time; waves less than 4 feet occur 63.3 percent of the time, and waves less than 6 feet occur 82.9 percent of the time. The largest storm events in the summer and early fall months are due to tropical disturbances, and the largest storm events in the late fall and winter months are due to northeasters.

An examination of table 4b shows an identical distribution of wave periods above 10 seconds as seen in the deepwater peak period summary presented in table 3b. A similar pattern between tables 3b and 4b is observed for wave periods below 10 seconds as well; the only difference between the two tables is caused by excluding the 'calm' westerly events from table 4b. The shallow-water summary presented in table 4b shows peak periods concentrated in the lower bands; 61.1 percent of the wave periods fall into the category of locally-produced 'wind waves' with periods below 8 seconds. An additional 33.7 percent of the wave record falls within the 8 to 14.99-second band, and the remaining 5.2 percent of events have peak periods in excess of 15 seconds. The longer wave periods generally correspond to larger wave heights, and many of the longer-period events shown in table 4b correspond to large storm waves which will break seaward of the 12-foot contour, and therefore will have less impact on nearshore littoral processes.

Table 4c shows how incident wave direction is altered as waves propagate from deepwater to shallow water depths. As described above, all waves incident from the westerly directions are removed from the nearshore wave record since they are propagating away from the shoreline. As the remaining waves propagate through progressively shallower waters the wave directions become aligned more perpendicular to the depth contours in accordance with Snells Law. The wave events summarized in table 4c are much more closely aligned to shore-normal than waves in greater water depths. The incident deepwater wave directions in table 3c originated from every sector but as seen in table 4c, the entire nearshore wave record is contained in five 22.5-degree angle bands, with 99 percent of the wave events contained in the central three bands.

At the 12-foot depth contour, the highest number of wave events originate from the easterly sector (78.75 – 101.24 degree) during every month of the year. For each month, the number of waves originating from this sector is nearly double the number of waves from all other sectors combined. As seen in the summary column on the right side of table 4c, waves originate from this central (easterly) band 59.9 percent of the time, while waves from the northeast sector (56.25 – 78.74 degrees) are the second most common occurrence, at 23.5 percent. Waves originating from the southeast (101.25 – 123.74 degrees) occur 15.6 percent of the time. Waves from the more extreme north and south directions occur much less frequently : 0.3 and 0.7 percent of the time, respectively.

Table 4c again demonstrates the seasonal variations of the South Florida wave environment. The shift of incident wave directions from the east-southeast during the summer months to a more east-northeasterly direction in the winter months is easily seen in this table. From May through August waves originate from the east and southeast sectors over 80 percent of the time during each of the summer months. Waves originate from the east and northeast directions over 80 percent of the time during the remaining winter months. The resulting longshore transport rates which have been calculated in previous studies verify this seasonal change in direction : most calculated gross transport rates are several times higher than the calculated net rates since the northerly sediment transport along the coastline during the summer months cancels out much of the southerly transport which occurs during the winter months.

Table 4d shows a summary of mean wave heights grouped by month and year throughout the 20-year period of record. It is seen that the shallow-water mean wave heights presented in this table are about 25 percent higher than the deepwater mean wave height presented in table 3d. This is in spite of the fact that 6.4 percent of the wave record has been removed and recorded as calms, since those waves propagate away from the shoreline. Seasonal effects are still evident however : mean wave heights average about 2 – 2.5 feet during the summer months, while mean wave heights of 4.5 – 5 feet are shown for some winter months. As seen in table 4a, the higher mean wave heights observed in the winter months are primarily due to a relatively small number of high-energy storm events. Table 4a also shows that the smaller-amplitude wave events occur the overwhelming majority of the time during every month of the year.

The main implication of this seasonal fluctuation of the wave environment is that any proposed plan of improvement for Bal Harbour must account for these variations in wave field characteristics. It is especially important to consider the seasonal changes in sediment transport magnitude and direction when considering the use of coastal structures which alter the flow of sediment along the coast. A more detailed analysis of the nearshore wave environment along the Bal Harbour shoreline will be provided in the Numerical Modeling section of this report, including local bathymetric effects and wave-current interactions.

Storm History.

This section will present an overview of significant storm events which have occurred since initial construction of the Bal Harbour project in 1975. Storms which have occurred from 1975 to the present time potentially affected the distribution of sediment along the coastline, possibly affecting the overall performance of the project. These effects may be reflected in the volumetric analyses which are presented later in this report.

Several hurricanes and strong northeasters have affected the Dade County coastal area since initial construction was begun in 1975. The following is a list of significant meteorological events which impacted the Federal shore protection project during that period. In some instances of severe storm damages the local sponsor requested assistance in repairing storm-induced erosion under the provisions of Public Law (PL) 84-99. PL84-99 is the primary mechanism for providing Federal funding (through FEMA) for repairing damages to Federal shore protection projects due to significant storm events. Only repairs to the design section are provided for under the provisions of PL84-99; damages to the advanced maintenance cannot be replaced under this authority.

1992 - Hurricane Andrew. Andrew was upgraded from a tropical storm to a hurricane on 22 August 1992 and struck the Bahamas on 23 August, causing widespread damage through the islands. The storm continued through the Bahamas on a near-westerly track, and made landfall as a strong category 4 hurricane about 25 miles south of downtown Miami near Homestead and Florida City, at 4:00 a.m. on 24 August. The National Hurricane Center estimated that at the time of landfall Andrew had sustained winds of 145 mph, with gusts to 175 mph. The storm center passed about 10 miles south of Key Biscayne, the southernmost point of the Atlantic shoreline in Dade County. Storm surges measured at the north and south ends of Key Biscayne were 10.1 and 10.6 feet, respectively. There were no offshore wave gages near the path of Hurricane Andrew, but a hindcast prepared by the U.S. Army Corps of Engineers' Waterways Experiment Station (WES) based on wind field data indicated that the maximum significant wave height was 22.6 feet, in water depths of 26 to 30 feet along the Miami Beach shoreline. In spite of the extreme wind, waves, and surge levels created by Andrew, damage to the BEC & HP was moderate to minimal along the length of the project, and the Federal project was attributed with saving 19.6 million dollars in upland damages.

Hurricane Andrew was classified as the third-strongest hurricane to make landfall in the U.S. in the 1900's. Total damages were estimated at between 15 and 20 billion dollars, with most of the upland structural damage resulting from high wind velocities. The normally high level of upland damage resulting from storm surge and wave impacts from landfalling hurricanes did not occur along Miami Beach and further to the north. It is believed that the relatively fast forward speed of the storm coupled with the protection provided by the Bahama Bank minimized the time that the coastline was exposed to large storm swells. These large swells broke well offshore, expending most of their energy and reducing damage potential by the time they reached the shoreline. Damage to the offshore coral reefs in water depths ranging from 30 to over 100 feet was extensive, and several artificial reefs in this depth range, including large shipwrecks, were moved considerable distances by storm swells during the passage of Hurricane Andrew.

Andrew impacted the Dade County shoreline in the interval between the June 1990 and June 1996 county-wide beach profile monitoring surveys, and the impacts of Hurricane Andrew are reflected in the volumetric analyses in the following sections of this report. A moderate amount of coastal erosion was attributed to the storm in a series of PL84-99 reports which were prepared to assess the impacts of Andrew. In these reports, a total of 2.8 miles of the 13.2-mile Federal project were determined to have been substantially damaged by Hurricane Andrew, and it was recommended that a total renourishment volume of 307,500 cubic yards of fill be placed along four affected areas of the project to repair storm damages to the design beach fill template. Hurricane Andrew was by far the strongest storm (in terms of magnitudes of windspeeds, storm surge, and wave heights) to impact the Dade County shoreline since construction of the Federal BEC & HP project.

1995 – Hurricane Erin. Tropical storm Erin strengthened into a hurricane over the lower Bahamas on 31 July 1995. Over the next three days Erin moved northwest over the Bahamas, eventually making landfall near Vero Beach, Florida on 2 August 1995. Although direct storm effects along the Dade County shoreline were minimal, large northeasterly swells were generated as Erin crossed the deep waters of the Gulf Stream prior to making landfall in Florida. Wave data obtained from NOAA buoy 42036 shows that Erin produced waves in excess of 15 feet, with a mean wave period of 7 to 9 seconds. These northerly swells caused some erosion along the southeast Florida coast, including Dade County. No PL84-99 assistance was requested.

1995 – Tropical Storm Jerry. Tropical storm Jerry formed on 22 August 1995 in the Gulf Stream, between the Florida Keys and Andros Island in the Bahamas. Jerry moved in a north-northwest direction, passing directly offshore of the Dade County coast, causing strong local winds and generating high waves and a minor storm surge along the Dade County shoreline. Some minor to moderate beach erosion was reported by the local sponsor, but damages were not of sufficient magnitude to request PL84-99 assistance.

1996 Northeaster. From 13-18 November 1996 a high pressure system off the east coast of the U.S. combined with a large low pressure system over the Caribbean to generate very strong easterly winds which affected the project area for several days. Sustained winds of over 60 knots were measured throughout Dade County during this period. These winds generated wave heights in excess of 10 feet which impacted the south Florida coastline for four days, causing significant erosion to portions of the Federal shore protection project in Dade County. At the request of the local sponsor, a PL84-99 report was prepared by the Jacksonville District to assess storm damages caused by this northeaster. Although it was determined that substantial damages had occurred along some reaches of the Federal project, these losses did not qualify for repairs under the provisions of PL84-99. This storm occurred in the interval between the June 1996 and June 1998 county-wide surveys which were used in volumetric analysis in the following sections of this report.

1998 – Hurricane Georges. In late September 1998 Hurricane Georges passed northward through the Straits of Florida, across the Florida Keys, and continued northward into the Gulf of Mexico. Winds generated by Georges impacted the Dade County area, generating storm waves which caused some coastal erosion. Gages at the Fowey Rocks meteorological station, located offshore of southern Dade County, recorded sustained winds of over 50 mph, with gusts up to 60 mph.

1999 – Hurricane Irene. Irene formed in the western Caribbean and traveled northward over Cuba, strengthening to hurricane force in the Florida Straits. The eye of the storm passed over Key West at 0900 on 15 October and made landfall along the southwest Florida coast several hours later. Irene was a weak Category I storm at landfall, with highest sustained winds of 85 mph. Sustained winds of 57 mph with gusts to 73 mph were measured at Fowey Rocks. The storm moved across the lower Florida peninsula, passing just to the northwest of Miami, and entered the Atlantic Ocean along the Palm Beach County coast. Although weak in intensity, Irene produced rainfall amounts up to 16 inches across south Florida, causing widespread flooding and power outages. Upon entering the Atlantic Ocean, Irene produced a large storm swell which caused some erosion of Federal shore protection projects in Dade, Broward, and Palm Beach counties. In addition to Hurricane Irene, Tropical Storm Dennis and Hurricane Floyd followed nearly identical tracks northward through the Bahamas, passing within a few hundred miles of the Dade County shoreline in August and September of 1999. Onshore winds generated by both storms created storm waves which impacted Dade County's shoreline for several days.

2000 – Hurricane Michelle. Michelle formed over Nicaragua on 29 October and began tracking to the northeast, passing over Cuba, and Andros Island and Eleuthera Island in the Bahamas. Although the storm did not pass directly over the southeast Florida coast, strong onshore winds generated by Michelle impacted the Dade County shoreline for several days. Sustained winds of 35-45 mph were measured at Miami Beach during this time, and winds up to 53 mph were measured at the meteorological station at Fowey Rocks. Storm surges of 1 – 3 feet were reported along the coast and high storm waves impacted the area for several days, causing significant beach erosion.

Conclusions. South Florida is located in a region of relatively high hurricane activity. An analysis of historical records indicates that 26 hurricanes have made landfall along the southeast Florida coast between 1900 and 2003, with 11 of these storms being classified as major hurricanes (categories 3, 4, or 5). Using this 103-year record as a statistical baseline, a hurricane has made landfall along the southeast Florida coast every 4.0 years on average, with a major hurricane directly impacting the area every 9.4 years.

ANALYSIS OF PHYSICAL DATA - REGIONAL

General.

This section of the report will examine physical data on a county-wide basis in order to accurately define the large-scale processes which affect the Bal Harbour shoreline. Later sections of this report will examine in greater detail the physical processes that affect only the Bal Harbour study area.

Survey data will be used as a basis for the volumetric and shoreline change measurements which will be presented in the first sections of this engineering analysis. Using Regional Sediment Management (RSM) guidelines, volumetric and shoreline position change rates will be calculated regionally (in this case county-wide) to determine shoreline changes over a broad area overlapping the Bal Harbour study area. Additional small-scale surveys will be used to provide more detail in the Bal Harbour study area as needed. Shoreline changes will be examined in this report over two distinct time intervals : prior to the beginning of construction of the Federal BEC & HP project (to establish pre-project "natural" conditions), and after completion of initial construction of the BEC & HP project (to establish the conditions presently observed with the Federal project in place). The "pre-project" interval is defined as prior to construction of the first segment of the Federal project in 1975. The "post-project" era begins with the completion of the last segment of the Federal project in 1988.

This section of the report will also describe some of the numerical modeling efforts which were conducted as part of this study, beginning with the detailed simulation of wave refraction using the STWAVE (STeady-state spectral WAVE) model. Wave-current interactions in the vicinity of Bakers Haulover Inlet will be modeled using STWAVE in conjunction with the ADCIRC (ADvanced CIRCulation) circulation model. Wave hindcast data described in previous sections of this report will be used as input to drive these numerical models. Once the physical processes in the project area are accurately defined, various alternative plans of improvement will be developed, and the numerical model GENESIS (GENEralized Model for the SImulation of Shoreline Change) will be used to evaluate the effects of these alternative plans on the study area shoreline. A final plan of improvement will then be selected based on these modeling results. Each of the tasks briefly described in this paragraph will be detailed in the following sections of this report.

Survey Database.

General. The basis for much of the engineering analysis contained in this report is a series of coastal surveys which have been performed periodically since the late 1800's. These surveys typically consist of elevations measured along shore-perpendicular lines which begin at pre-established survey monuments located near the seawall line and extend seaward for several thousand feet. Surveyed elevations are measured along these shore-perpendicular lines at intervals of 25 feet or less, and the data obtained along these lines are used to compute shoreline change rates and volumetric changes along the shoreline. In more recent times these surveys have been used as a source of bathymetric data for numerical shoreline modeling.

Historical Surveys. Surveys performed prior to BEC & HP project construction were based on an older series of survey monuments which are no longer used. These monuments are numbered consecutively from 1 through 19 extending from Government Cut northward to Bakers Haulover Inlet, and 20 through 26 extending from Bakers Haulover Inlet northward to the Dade/Broward county line. The county-wide surveys performed in 1867, 1883, 1919, 1927/28, and 1961 are all based on this older system of survey monumentation.

Other separate sets of survey monuments were established by the Corps of Engineers, the county, the city of Miami Beach and others throughout the early and mid-1900's along the Dade County shoreline. Surveys were performed sporadically prior to construction of the

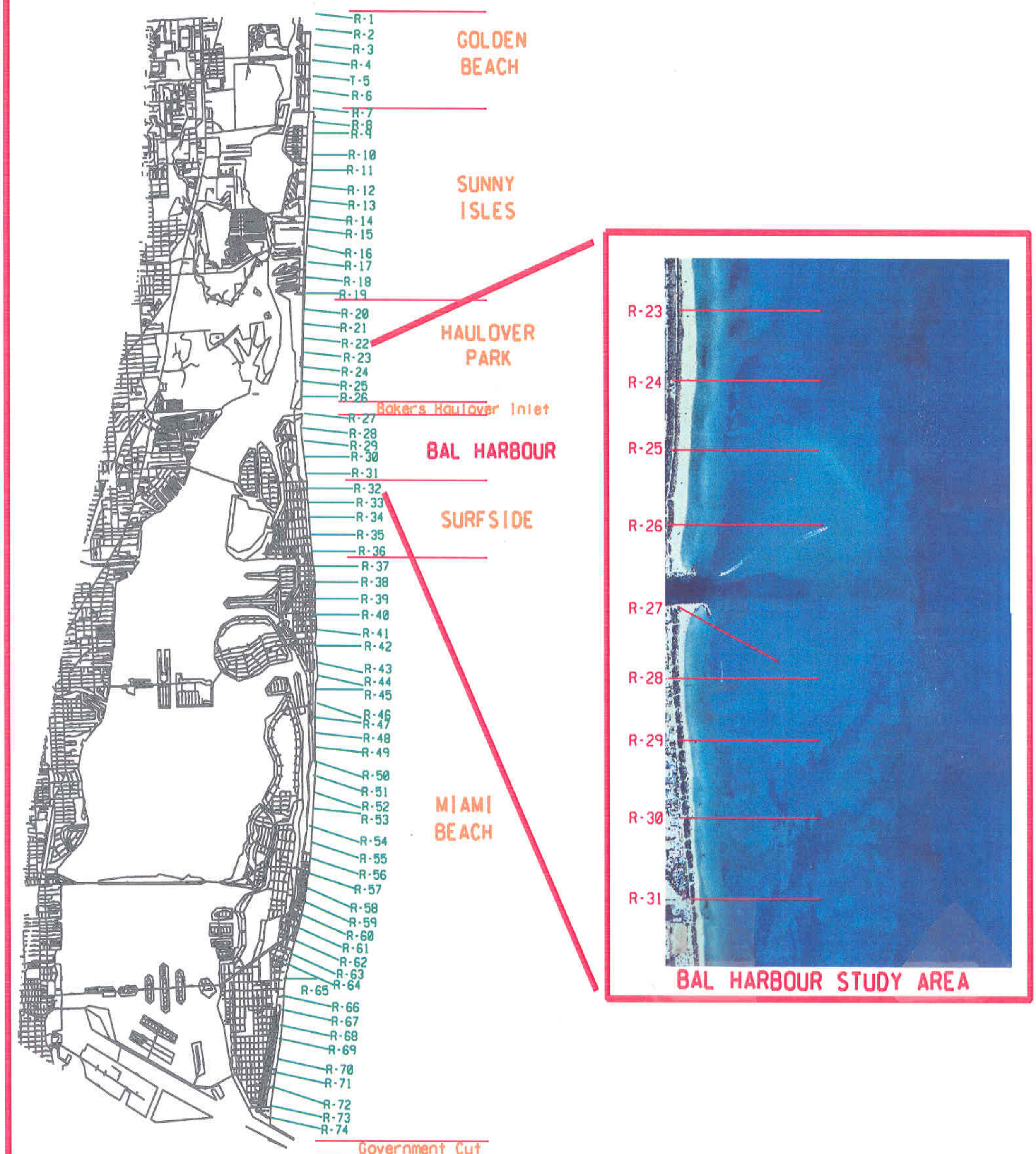
Federal project in the 1970's, and consecutive surveys often were based on different sets of monuments, which made volumetric comparisons difficult.

DNR Survey Control. In the interest of maintaining a consistent database the DNR monumentation system was adopted as a standard in the late 1980's. This state-wide system of survey monuments was established by the Florida Department of Natural Resources (now Department of Environmental Protection) in 1974. These DNR monuments are now are used as the control points for all beach profile surveys along the Dade County shoreline. The monuments are positioned approximately every 1,000 feet along the Dade County Atlantic coast, beginning at the Dade/Broward county line (DNR-1), and extending to the southern tip of Key Biscayne (DNR-113). The Federal BEC & HP project extends from DNR-7 at the north end of Sunny Isles southward to DNR-74 at Government Cut.

These DNR monuments are commonly used to define positions along the shoreline. For example, the Bal Harbour shoreline extends from Bakers Haulover Inlet southward to 96th Street, which corresponds to monuments DNR-27 to DNR-31. A map of the Dade County shoreline showing DNR monuments along the length of the Federal project is shown in figure 9. A close-up view of monument locations along the Bal Harbour study area is also shown in this figure. For brevity, the monument designations are often abbreviated as "R" instead of "DNR". This abbreviated designation will be used throughout this report. Three of the county-wide surveys used in the following sections of this report (performed in 1990, 1996, and 1998) are based on the monuments shown in figure 9.

Two of the more recent surveys of the Dade County shoreline have been performed using lidar technology. LIDAR is an acronym for LIght Detection And Ranging, and consists of an aircraft-mounted laser which is flown over the survey area, firing rapid laser pulses and measuring the time required for each laser pulse to be reflected from the ground back to the aircraft. These measured times, combined with precise positioning information of the aircraft, are then converted to surface elevations. Lidar results in much greater coverage of the survey area than the traditional beach profiling methods, with elevation measurements spaced as little as an average of 4 meters apart over the entire survey area. Two lidar surveys of the Dade County shoreline were taken in 2000 and 2002. Each of these surveys covered the entire Dade County BEC & HP north to south and provided the most detailed picture ever obtained of the complex bathymetry offshore of Dade County's shorelines. These lidar surveys were used to define offshore bathymetry for the numerical modeling which was conducted as part of this study.

Comparative beach profiles are provided in Appendix E for each DNR survey monument in the study area (R-19 through R-36). These profiles are based on the five county-wide surveys taken in 1990, 1996, 1998, 2000, and 2002. Since the lidar surveys consist of a random scatter of data points, profiles were obtained by first creating 3-dimensional digital terrain models (dtm's), then digitally cutting profiles across the dtm's at each DNR monument at the correct azimuths.



DNR MONUMENT LAYOUT - DADE COUNTY

FIGURE 9.

Shoreline Position Change Analysis.

Pre-Project MHW Changes. County-wide changes in the position of the mean high water (mhw) line were measured during the period prior to construction of the Federal project, based on surveys taken in 1867, 1883, 1919, 1927-28, and 1961. A summary of mhw changes as measured from these surveys was developed in the 1975 Dade County General Design Memorandum (reference 1b) and is presented in table 5. Note that a different set of survey monuments was used during this period; approximate present-day DNR profile locations are provided in parentheses. Under this old system of survey monumentation, profiles 18 and 19 were the only profiles that fell within the city limits of Bal Harbour. Profile 18 was located near the south end of Bal Harbour (near R-30), and profile 19 was located at the north end of Bal Harbour, adjacent to the south side of Bakers Haulover Inlet (near R-27). Survey data is included in table 5 for the entire reach of shoreline extending from the north county line southward to Government Cut, to allow comparison between shoreline changes at Bal Harbour and the rest of the Dade County shoreline.

As seen in table 5, the shoreline primarily advanced between Bakers Haulover Inlet and Government Cut between 1867 and 1919. The average shoreline advance along this reach during this 52-year interval was 168 feet, with individual profiles showing advances of from 20 to 490 feet. The average shoreline advance measured along Bal Harbour during this period was 210 feet. This value was based on measurements of shoreline advances of 380 feet and 40 feet at profiles 18 and 19, respectively. Only three profiles showed shoreline recession during this period, averaging -47 feet near the center of Miami Beach. The mhw advance measured along this reach of coast during this time interval represents the greatest (and most consistent) shoreline advance measured in the history of Dade County's beach surveys. Much of this advance was attributed to the construction of the jetties at Government Cut beginning in 1903, which impounded large volumes of material north of the north jetty due to the net southerly littoral transport of sediment along the coast. North of Bakers Haulover Inlet, profiles taken in 1883 and 1919 indicate an average shoreline advance of 95 feet throughout Haulover Park during this 36-year period. Shoreline recessions (or no change) were measured throughout Sunny Isles and Golden Beach during the period from 1883 to 1927-28. The average recession along Sunny Isles and Golden Beach was -59 feet during this period, based on seven profile measurements ranging from 0 to –200 feet over this 44-year timeframe.

TABLE 5
SUMMARY OF HISTORIC MHW SHORELINE CHANGES
DADE COUNTY, FLORIDA

Profile	Location	Approx. DNR	1867-1919 Recession	1867-1919 Advance	1919-1927/28 Recession	1919-1927/28 Advance	1927/28 - 1961 Recession	1927/28 - 1961 Advance	1867 - 1961 Recession	1867 - 1961 Advance
1a	Miami Beach	R-74		490	-140			200		550
1	"	R-71		310	-80			120		350
2	"	R-69		280	-30			170		420
3	"	R-67		180	-80		-40			60
4	"	R-64		100	-60		-40		0	0
5	"	R-61	-40		-50		0	0	-90	
6	"	R-59		150	-180			60		30
7	"	R-56		60	-200			40	-100	
8	"	R-54	-50		-80		-50		-180	
9	"	R-52	-50		-130			100	-80	
10	"	R-49		20	-40			20	0	0
11	"	R-47		80	-110		0	0	-30	
12	"	R-44		200	-160		0	0		40
13	"	R-42		100	-110			70		60
14	"	R-39		120	-150			50		20
15	"	R-37		200	-110			110		200
16	Surfside	R-35		350	-120			60		290
17	Surfside	R-32		450	-140			60		370
18	**Bal Harbour**	**R-30**		**380**	**-60**		**-40**			**280**
19	**Bal Harbour**	**R-27**		**40**	**-100**		**-20**		**-80**	
			AVERAGE:	+168'	AVERAGE:	-107'	AVERAGE:	+52'	AVERAGE:	+106'
			1883 to 1919		**1919-1927/28**		**1927/28 - 1961**		**1883 - 1961**	
19A	Haulover Park	R-26		50	-140		0	0	-90	
19B	"	R-24		70	-190		-20		-140	
19C	"	R-21		100	-160		0	0	-60	
19D	"	R-19		160	-160		-40		-40	
			AVERAGE:	+95'	AVERAGE:	-162'	AVERAGE:	-15'	AVERAGE:	-83'
			1883 to 1927/28		**1919-1927/28**		**1927/28 - 1961**		**1883 - 1961**	
20	Sunny Isles	R-16	-50		N/A			100		50
21	"	R-14	0	0	N/A		0	0	0	0
22	"	R-11	0	0	N/A			130		130
23	"	R-9	-70		N/A			40	-30	
24	Golden Beach	R-6	-200		N/A			200	0	0
25	"	R-4	-40		N/A			40	0	0
26	"	R-1	-50		N/A		0	0	-50	
			AVERAGE:	-59'	AVERAGE:	N/A	AVERAGE:	+73'	AVERAGE:	+14'

A strong erosive trend appears during the 1919 to 1927-28 period. Shoreline recession was noted at every profile between Government Cut and Bakers Haulover Inlet during this time. Shoreline retreat ranged from -30 to -200 feet, with an average recession of -107 feet along this reach during the 8-year period. The average shoreline recession along Bal Harbour during this period was -80 feet, based on measured recessions of -60 feet and -100 feet at profiles 18 and 19, respectively. Recession was also noted north of Bakers Haulover Inlet, with an average recession of -162 feet throughout Haulover Park. No profiles were surveyed through Sunny Isles of Golden Beach during this timeframe. The consistent shoreline recession along the surveyed portion of the county's shoreline may be due to two factors : the occurrence of the devastating hurricane of 1926 which displaced large volumes of beach sediment landward, and the construction of Bakers Haulover Inlet in 1925, which created a disruption to the littoral transport of sediment along the coast.

The overall trend again reversed between 1927-28 and 1961, with shoreline advance being measured at most profiles between Government Cut and Bakers Haulover Inlet. Advances in the mhw line of 0 to 200 feet were measured along this reach, with an average value of 52 feet of advance during this 34-year period. The largest advances were near the Government Cut north jetty, and were likely the result of continued impoundment north of the jetty. Five profiles indicated recession along this reach during this time interval, averaging –38 feet. Records indicate that Bakers Haulover Inlet was dredged 11 times between 1937 and 1961. Records of the 8 maintenance dredging events between 1937 and 1944 show that an average of 17,000 cubic yards per year were removed from the inlet, but the disposal areas for these events are not indicated. Records from the three dredging events from 1955-61 indicate placement along Haulover Park. In addition, beach fill placements of 180,000 and 32,040 cubic yards were placed along Haulover Park in 1960 and 61, respectively, and beach fills of 86,000 and 25,000 cubic yards were placed along Bal Harbour in the same years. In spite of these beach fill placements, an average mhw recession of -30 feet was measured at Bal Harbour during this period (-40 feet and –20 feet at profiles 18 and 19, respectively). This recession was likely due to the interruption of sediment transport southward around Bakers Haulover Inlet. An average recession of –15 feet was measured along Haulover Park, and further to the north an average of 73 feet of advance was measured along the Sunny Isles and Golden Beach shorelines during the same period. Shoreline positions at several profile locations may have been influenced by the extensive construction of groins and seawalls during this time period. Common practice at the time involved constructing seawalls seaward of the existing waterline, then backfilling the structure. This would appear as an apparent shoreline advance in the monitoring surveys, and it is not known which profiles may have been affected in this manner.

A summary of mhw shoreline changes over the period from 1867 (or 1883 for the north county) through 1961 is given on the far right side of table 5. As shown in this summary, the region from Government Cut to Bakers Haulover Inlet advanced an average of 106 feet over the time period 1867 to 1961 (+1.1 ft/yr). During this 94-year period the region generally showed the greatest advances at either end (especially at the south end), with shoreline recession generally noted in the central region. The shoreline advances at the south end of this reach can be attributed to the impoundment of large volumes of material following completion of the Government Cut north jetty, while the advances at the north end may be attributed to the placement of numerous beachfills along the Bal Harbour shoreline during this period and to the impoundment of some northward-transported sediment along the south side of the inlet during the summer months. The Bal Harbour shoreline advanced by an average of 100 feet during this 94-year period (+280 feet and –80 feet at profiles 18 and 19, respectively), resulting in an average change rate of +1.1 ft/yr. The shoreline along Haulover Park receded by an average of –83 feet over the 78-year period from 1883 to 1961 (-1.1 ft/yr). The 7 profiles surveyed through the towns of Sunny Isles and Golden Beach showed an average accretion of +14 feet over the same 78-year period (+0.2 ft/yr). In each case these averages should be used cautiously due to the large degree of variability among change rates measured at individual profiles.

Prior to construction of Bakers Haulover Inlet the shorelines adjacent to the inlet at Bal Harbour and Haulover Park appeared to be stable or accretionary. Following construction of the inlet in 1925 these beaches showed more of a tendency toward erosion, assumed to be primarily the result of the interruption of littoral transport. The effects are more pronounced closer to the inlet, as seen by comparing the shoreline change rates between profiles 18 and 19. Profile 18 (located near the south end of Bal Harbour) advanced 280 feet over the 94-year period 1867-1961, while profile 19 (located at the north end of Bal Harbour) receded by –80 feet during the same period. When annualized, these rates equate to +3.0 feet per year of advance at the southern end of Bal Harbour, and –0.85 feet per year of recession at the northern end.

An average value of shoreline change along the entire county was calculated by summing the values contained in the summary portion of table 5. The average value of shoreline change was +61 feet of mhw shoreline advance for the period 1867 (1883) through 1961. However, due to the inherent errors introduced into the shoreline database from the seaward displacement of the mhw shoreline by the seawall construction as described above, caution should be exercised when using this data to determine long-term pre-project shoreline changes. Over the 94-year pre-project survey interval, the southern end of Bal Harbour advanced at a much greater rate than the county-wide average, while the northern end of Bal Harbour eroded at a relatively moderate rate.

The placement of beach fills during this pre-construction period also distorts the ‘true’ mhw changes due to natural processes only. Several beach fill placements were completed from 1867 to 1961, but insufficient data exists on exact fill placement locations, times of placement, and dimensions of beach fills. It is therefore not possible to remove the effects of these beach fill placements from the mhw database. Fortunately however, the volumes of beach fill placement were relatively low, with long time intervals between successive fill placements. These factors would tend to minimize the effect of these fills on the ‘natural’ shoreline changes.

Post-Project MHW Changes. The placement of large volumes of fill along the length of the Federal BEC & HP project from 1975 (beginning of Bal Harbour construction) through 1988 (completion of Sunny Isles construction) caused a large degree of apparent shoreline advance along the Dade County shoreline which is separate from the natural processes that this analysis attempts to define. This period of initial construction will therefore be omitted from this analysis because of the obvious effects of large-scale beach fill placements on the mhw positions, and due to the lack of county-wide surveys during this period. The shoreline change analysis in this section will include the entire length of the Federal project in order to allow a comparison between shoreline changes along Bal Harbour and large-scale trends along the adjacent shorelines. The period of analysis for post-construction shoreline position changes will therefore extend from 1990 to 2002, and will be based on the county-wide surveys performed in 1990, 1996, 1998, 2000, and 2002.

The county-wide surveys conducted in 1990, 1996, and 1998 were based on DNR beach profiles; the surveys conducted in 2000 and 2002 were performed using lidar. Beach profiles from each of the three county-wide surveys from 1990 to 1998 were plotted using CADD software, and mhw distances were measured from these profiles. The lidar surveys were also entered into a CADD system, converted into three-dimensional digital terrain models, and profiles were then cut across the dtm's along DNR azimuths to allow measurement of the mhw shoreline positions in the same manner as the three beach profile surveys. The resulting measurements are shown in table 6. Table 6 consists of three sets of columns, which show (from left to right) distances measured from the ECL to the mhw line at each profile, differences in mhw positions between successive surveys at each profile, and differences between mhw positions with beach fill effects removed.

The first data set (left side of table 6) provides the horizontal distances from the ECL to the mean high water line at each profile for each of the five post-construction surveys. These values provide a measure of actual beach width, and provide an indication of the condition of the project. The hypothetical minimum beach widths required from ECL to mhw, based on the project's design templates are 148 feet along Sunny Isles (R7-19), 178 feet along Haulover Park (R20-26), and 223 feet from Bakers Haulover Inlet to Government Cut (R27-74), which includes Bal Harbour. In practice, these distances have proven to be excessive, because beach fill tends to stabilize at steeper slopes than the design profile. The design profile for the three templates consists of a 1v : 20h slope from berm to mhw, then 1v : 40h from mhw to existing bottom. Actual project monitoring suggests that a front slope of 1v 11h may be more realistic when using an offshore borrow source. Based on the steeper slope of 1v : 11h observed along the length of the project, the minimum widths from ECL to mhw become approximately 90 feet along Sunny Isles, 120 along Haulover Park, and 165 feet along Bal Harbour, Surfside, and Miami Beach.

TABLE 6

Post-Project MHW Changes

DNR MON	Distances, ECL to MHW					As Measured From Surveys									
	1990	1996	1998	2000	2002	1990 vs 96		1996 vs 98		1998 vs 2000		2000 vs 2002		1990 vs 2002	
						Total(ft)	Avg(ft/yr)	Total(ft)	Avg(ft/yr)	Total(ft)	Avg(ft/yr)	Total(ft)	Avg(ft/yr)	Total(ft)	Avg(ft/yr)
1	169	161	161	280	217	-8	-1.3	0	0	119	59.5	-63	-26.25	48	3.9
2	142	135	144	134	113	-7	-1.2	9	4.5	-10	-5	-21	-8.8	-29	-2.3
3	156	159	154	144	162	3	0.5	-5	-2.5	-10	-5	18	7.5	6	0.5
4	175	241	258	276	250	66	11.0	17	8.5	23	11.5	-26	-10.8	75	6.0
5	97	145	131	147	164	48	8.0	-14	-7	16	8	17	7.1	67	5.4
6	123	155	171	181	202	32	5.3	16	8	10	5	21	8.8	79	6.4
7	35	24	18	3	115	-11	-1.8	-6	-3	-15	-7.5	112	46.7	80	6.5
8	22	17	82	28	163	-5	-0.8	65	32.5	-54	-27	135	56.3	141	11.4
9	161	23	107	56	213	-138	-23.0	84	42	-51	-25.5	157	65.4	52	4.2
10	81	43	98	106	166	-38	-6.3	55	27.5	8	4	60	25.0	85	6.9
11	135	54	107	65	160	-81	-13.5	53	26.5	-42	-21	95	39.6	25	2.0
12	145	78	101	107	171	-67	-11.2	23	11.5	6	3	64	26.7	26	2.1
13	118	82	86	65	127	-36	-6.0	4	2	-21	-10.5	62	25.8	9	0.7
14	117	86	90	116	127	-31	-5.2	4	2	26	13	11	4.6	10	0.8
15	155	123	123	86	173	-32	-5.3	0	0	-37	-18.5	87	36.3	18	1.5
16	147	88	109	113	138	-59	-9.8	21	10.5	4	2	25	10.4	-9	-0.7
17	206	109	126	102	164	-97	-16.2	17	8.5	-24	-12	62	25.8	-42	-3.4
18	145	90	102	95	133	-55	-9.2	12	6	-7	-3.5	38	15.8	-12	-1.0
19	106	52	65	81	102	-54	-9.0	13	6.5	16	8	21	8.8	-4	-0.3
20	175	131	133	121	160	-44	-7.3	2	1	-12	-6	39	16.3	-15	-1.2
21	153	109	126	127	148	-44	-7.3	17	8.5	1	0.5	21	8.8	-5	-0.4
22	147	121	136	125	145	-26	-4.3	15	7.5	-11	-5.5	20	8.3	-2	-0.2
23	171	146	133	172	176	-25	-4.2	-13	-6.5	39	19.5	4	1.7	5	0.4
24	177	156	202	199	175	-21	-3.5	46	23	-3	-1.5	-24	-10.0	-2	-0.2
25	175	197	172	197	190	22	3.7	-25	-12.5	25	12.5	-7	-2.9	15	1.2
26	148	153	110	161	152	5	0.8	-43	-21.5	51	25.5	-9	-3.8	4	0.3
27	**447**	**284**	**331**	**289**	**314**	**-163**	**-27.2**	**47**	**23.5**	**-64**	**-32**	**25**	**10.4**	**-133**	**-10.7**
28	**183**	**79**	**155**	**112**	**95**	**-104**	**-17.3**	**76**	**38**	**-42**	**-21**	**-17**	**-7.1**	**-88**	**-7.1**
29	**289**	**184**	**259**	**170**	**145**	**-105**	**-17.5**	**75**	**37.5**	**-42**	**-21**	**-25**	**-10.4**	**-51**	**-4.1**
30	**171**	**115**	**179**	**184**	**161**	**-56**	**-9.3**	**64**	**32**	**-34**	**-17**	**-23**	**-9.6**	**-80**	**-6.5**
31	**271**	**166**	**141**	**233**	**200**	**-105**	**-17.5**	**-25**	**-12.5**	**93**	**46.5**	**-33**	**-13.8**	**-71**	**-5.7**
32	217	163	161	279	249	-54	-9.0	-2	-1	118	59	-30	-12.5	32	2.6
33	207	187	138	311	256	-20	-3.3	-49	-24.5	173	86.5	-55	-22.9	49	4.0
34	212	158	171	324	255	-54	-9.0	13	6.5	153	76.5	-69	-28.8	43	3.5
35	195	156	111	285	220	-39	-6.5	-45	-22.5	174	87	-65	-27.1	25	2.0
36	225	143	125	267	212	-82	-13.7	-18	-9	142	71	-55	-22.9	-13	-1.0
37	241	197	168	220	195	-44	-7.3	-29	-14.5	52	26	-25	-10.4	-46	-3.7
38	269	208	235	187	181	-61	-10.2	27	13.5	-48	-24	-6	-2.5	-88	-7.1
39	258	244	197	163	181	-14	-2.3	-47	-23.5	-34	-17	18	7.5	-77	-6.2
40	267	218	196	210	205	-49	-8.2	-22	-11	14	7	-5	-2.1	-62	-5.0
41	223	204	201	195	172	-19	-3.2	-3	-1.5	-6	-3	-23	-9.6	-51	-4.1
42	240	220	220	192	206	-20	-3.3	0	0	-28	-14	14	5.8	-34	-2.7
43	179	153	160	140	176	-26	-4.3	7	3.5	-20	-10	36	15.0	-3	-0.2
44	178	134	151	129	173	-44	-7.3	17	8.5	-22	-11	44	18.3	-5	-0.4
45	216	178	146	146	190	-38	-6.3	-32	-16	0	0	44	18.3	-26	-2.1
46	253	184	200	144	198	-69	-11.5	16	8	-56	-28	54	22.5	-55	-4.4
47	336	311	283	249	298	-25	-4.2	-28	-14	-34	-17	49	20.4	-38	-3.1
48	386	353	345	331	326	-33	-5.5	-8	-4	-14	-7	-5	-2.1	-60	-4.8
49	306	280	268	263	231	-26	-4.3	-12	-6	-5	-2.5	-32	-13.3	-75	-6.0
50	233	165	149	158	146	-68	-11.3	-16	-8	9	4.5	-12	-5.0	-87	-7.0
51	294	243	224	238	205	-51	-8.5	-19	-9.5	14	7	-33	-13.8	-89	-7.2
52	303	262	242	263	231	-41	-6.8	-20	-10	21	10.5	-32	-13.3	-72	-5.8
53	316	261	307	296	264	-55	-9.2	46	23	-11	-5.5	-32	-13.3	-52	-4.2
54	251	195	314	275	210	-56	-9.3	119	59.5	-39	-19.5	-65	-27.1	-41	-3.3
55	185	122	262	182	137	-63	-10.5	140	70	-80	-40	-45	-18.8	-48	-3.9
56	204	151	272	186	148	-53	-8.8	121	60.5	-86	-43	-38	-15.8	-56	-4.5
57	205	134	260	192	142	-71	-11.8	126	63	-68	-34	-50	-20.8	-63	-5.1
58	196	128	217	186	175	-68	-11.3	89	44.5	-31	-15.5	-11	-4.6	-21	-1.7
59	167	83	125	139	406	-84	-14.0	42	21	21	10.5	267	111.3	239	19.3
60	253	222	224	211	308	-31	-5.2	2	1	-13	-6.5	97	40.4	55	4.4
61	264	254	257	252	275	-10	-1.7	3	1.5	-5	-2.5	23	9.6	11	0.9
62	283	306	268	296	301	23	3.8	-38	-19	28	14	5	2.1	18	1.5
63	446	465	462	447	454	19	3.2	-3	-1.5	-15	-7.5	7	2.9	8	0.6
64	524	544	524	557	500	20	3.3	-20	-10	33	16.5	-57	-23.8	-24	-1.9
65	453	500	503	511	509	47	7.8	3	1.5	8	4	-2	-0.8	56	4.5
66	405	448	494	472	458	43	7.2	46	23	-22	-11	-14	-5.8	53	4.3
67	382	384	427	438	409	2	0.3	43	21.5	11	5.5	-29	-12.1	27	2.2
68	388	355	363	377	330	-33	-5.5	8	4	14	7	-47	-19.6	-58	-4.7
69	397	440	425	415	353	43	7.2	-15	-7.5	-10	-5	-62	-25.8	-44	-3.5
70	413	389	390	373	358	-24	-4.0	1	0.5	6	3	-15	-6.3	-55	-4.4
71	312	297	313	328	329	-15	-2.5	16	8	15	7.5	1	0.4	17	1.4
72	271	257	252	310	314	-14	-2.3	-5	-2.5	58	29	4	1.7	43	3.5
73	182	287	290	312	272	105	17.5	3	1.5	22	11	-40	-16.7	90	7.3
74	217	224	202	355	303	7	1.2	-22	-11	159	79.5	-52	-21.7	86	6.9
					AVG:	-32.2	-5.4	14.0	7.0	7.5	3.8	6.8	2.8	-3.9	-0.3

DNR MON	With Beach Fills Removed									
	1990 vs 96		1996 vs 98		1998 vs 2000		2000 vs 2002		1990 vs 2002	
	Total(ft)	Avg(ft/yr)	Total(ft)	Avg(ft/yr)	Total(ft)	Avg(ft/yr)	Total(ft)	Avg(ft/yr)	Total(ft)	Avg(ft/yr)
1	-8	-1.3	0	0	119	59.5	-63	-26.25	48	3.9
2	-7	-1.2	9	4.5	-10	-5	-21	-8.8	-29	-2.3
3	3	0.5	-5	-2.5	-10	-5	18	7.5	6	0.5
4	66	11.0	17	8.5	18	9	-26	-10.8	75	6.0
5	48	8.0	-14	-7	16	8	17	7.1	67	5.4
6	32	5.3	16	8	10	5	21	8.8	79	6.4
7	-81	-13.5	-101	-50.5	-15	-7.5	-72	-30.0	-269	-21.7
8	-75	-12.5	-30	-15	-54	-27	-49	-20.4	-208	-16.8
9	-208	-34.7	-11	-5.5	-51	-25.5	-27	-11.3	-297	-24.0
10	-38	-6.3	-40	-20	8	4	-124	-51.7	-194	-15.6
11	-81	-13.5	28	14	-42	-21	-89	-37.1	-184	-14.8
12	-67	-11.2	23	11.5	6	3	-120	-50.0	-158	-12.7
13	-36	-6.0	4	2	-21	-10.5	-122	-50.8	-175	-14.1
14	-31	-5.2	4	2	26	13	-173	-72.1	-174	-14.0
15	-32	-5.3	-25	-12.5	-37	-18.5	-97	-40.4	-191	-15.4
16	-59	-9.8	-4	-2	4	2	-159	-66.3	-218	-17.6
17	-97	-16.2	-8	-4	-24	-12	-122	-50.8	-251	-20.2
18	-55	-9.2	12	6	-7	-3.5	-146	-60.8	-196	-15.8
19	-54	-9.0	13	6.5	16	8	-163	-67.9	-188	-15.2
20	-94	-15.7	2	1	-12	-6	39	16.3	-65	-5.2
21	-94	-15.7	17	8.5	1	0.5	21	8.8	-55	-4.4
22	-76	-12.7	15	7.5	-11	-5.5	20	8.3	-52	-4.2
23	-75	-12.5	-13	-6.5	39	19.5	4	1.7	-45	-3.6
24	-71	-11.8	46	23	-3	-1.5	-24	-10.0	-52	-4.2
25	-28	-4.7	-25	-12.5	25	12.5	-7	-2.9	-35	-2.8
26	5	0.8	-43	-21.5	51	25.5	-9	-3.8	4	0.3
27	**-163**	**-27.2**	**47**	**23.5**	**-42**	**-21**	**25**	**10.4**	**-133**	**-10.7**
28	**-104**	**-17.3**	**1**	**0.5**	**-43**	**-21.5**	**-17**	**-7.1**	**-163**	**-13.1**
29	**-105**	**-17.5**	**0**	**0**	**-34**	**-17**	**-25**	**-10.4**	**-126**	**-10.2**
30	**-56**	**-9.3**	**-11**	**-5.5**	**61**	**30.5**	**-23**	**-9.6**	**-155**	**-12.5**
31	**-105**	**-17.5**	**-100**	**-50**	**92**	**46**	**-33**	**-13.8**	**-146**	**-11.8**
32	-54	-9.0	-2	-1	-132	-66	-30	-12.5	-218	-17.6
33	-20	-3.3	-49	-24.5	-77	-38.5	-55	-22.9	-201	-16.2
34	-54	-9.0	13	6.5	-97	-48.5	-69	-28.8	-207	-16.7
35	-39	-6.5	-45	-22.5	-76	-38	-65	-27.1	-225	-18.1
36	-82	-13.7	-18	-9	-108	-54	-55	-22.9	-263	-21.2
37	-44	-7.3	-29	-14.5	52	26	-25	-10.4	-46	-3.7
38	-61	-10.2	27	13.5	-48	-24	-6	-2.5	-88	-7.1
39	-14	-2.3	-47	-23.5	-34	-17	18	7.5	-77	-6.2
40	-49	-8.2	-22	-11	14	7	-5	-2.1	-62	-5.0
41	-19	-3.2	-3	-1.5	-6	-3	-23	-9.6	-51	-4.1
42	-20	-3.3	0	0	-28	-14	14	5.8	-34	-2.7
43	-26	-4.3	7	3.5	-20	-10	36	15.0	-3	-0.2
44	-44	-7.3	17	8.5	-47	-23.5	-256	-106.7	-330	-26.6
45	-38	-6.3	-32	-16	-25	-12.5	-256	-106.7	-351	-28.3
46	-69	-11.5	16	8	-56	-28	-246	-102.5	-355	-28.6
47	-25	-4.2	-28	-14	-34	-17	49	20.4	-38	-3.1
48	-33	-5.5	-8	-4	-14	-7	-5	-2.1	-60	-4.8
49	-26	-4.3	-12	-6	-5	-2.5	-32	-13.3	-75	-6.0
50	-68	-11.3	-16	-8	9	4.5	-12	-5.0	-87	-7.0
51	-51	-8.5	-19	-9.5	14	7	-33	-13.8	-89	-7.2
52	-41	-6.8	-20	-10	21	10.5	-32	-13.3	-72	-5.8
53	-55	-9.2	-184	-92	-11	-5.5	-32	-13.3	-282	-22.7
54	-286	-47.7	-136	-68	-39	-19.5	-65	-27.1	-526	-42.4
55	-293	-48.8	-115	-57.5	-80	-40	-45	-18.8	-533	-43.0
56	-283	-47.2	-134	-67	-86	-43	-38	-15.8	-541	-43.6
57	-301	-50.2	-129	-64.5	-68	-34	-50	-20.8	-548	-44.2
58	-93	-15.5	-166	-83	-31	-15.5	-11	-4.6	-301	-24.3
59	-109	-18.2	17	8.5	21	10.5	260	108.3	189	15.2
60	-31	-5.2	-23	-11.5	-13	-6.5	97	40.4	30	2.4
61	-10	-1.7	3	1.5	-5	-2.5	23	9.6	11	0.9
62	23	3.8	-38	-19	28	14	5	2.1	18	1.5
63	19	3.2	-3	-1.5	-15	-7.5	7	2.9	8	0.6
64	20	3.3	-20	-10	33	16.5	-57	-23.8	-24	-1.9
65	47	7.8	3	1.5	8	4	-2	-0.8	56	4.5
66	43	7.2	46	23	-22	-11	-14	-5.8	53	4.3
67	2	0.3	43	21.5	11	5.5	-29	-12.1	27	2.2
68	-33	-5.5	8	4	14	7	-47	-19.6	-58	-4.7
69	43	7.2	-15	-7.5	-10	-5	-62	-25.8	-44	-3.5
70	-24	-4.0	1	0.5	-17	-8.5	-15	-6.3	-55	-4.4
71	-15	-2.5	16	8	15	7.5	1	0.4	17	1.4
72	-14	-2.3	-5	-2.5	-122	-61	4	1.7	-137	-11.0
73	105	17.5	3	1.5	-158	-79	-40	-16.7	-90	-7.3
74	7	1.2	-22	-11	-27	-13.5	-52	-21.7	-94	-7.6
AVG:	-52.2	-8.7	-17.5	-8.8	-16.1	-8.1	-37.8	-15.7	-124.8	-10.1

Based on these values, a comparison with the actual measured distances in table 6 shows that the design template remained in place along much of the project in the period following construction of the Federal project. However, some reaches of the project were consistently below the design width : the northernmost 3,000 feet of Sunny Isles, and limited areas of Miami Beach near R-45 (63rd Street) and R-59 (32nd Street). Each of these three areas have been noted to be among the most erosive regions of the project since project completion in the 1980's. As a result, frequent renourishments of these areas have been required during the postconstruction period (see previous section on beach fill history).

In regards to Bal Harbour, note that berm widths generally exceeded the minimum 165-foot design width at most profiles, during each survey period. The 1990 and 1998 surveys were each taken shortly after the completion of beach renourishment projects and reflect the added berm width of these renourishments. The 1998 beach renourishment suffered very high losses following construction however, and some of the measured berm widths were already below minimum at the time of that survey. During the 1996 survey, beach widths were mostly near or below the minimum design berm, and in the 2000 and 2002 lidar surveys some relatively minor erosion into the design berm is noted at some profiles.

Several trends along Bal Harbour are noted from analysis of this portion of table 6. First, in each of the five surveys, the most eroded portion of Bal Harbour occurs near the north end of the fill in the vicinity of R-28, which is located about 1,000 feet south of the inlet. Beach widths generally increase proceeding southward from this point into Surfside. Secondly, the effects of each erosion/renourishment cycle can be seen in the berm width measurements along each profile : berm widths are large following the 1990 renourishment, erosion occurs through the 1996 survey, renourishment again increases berm widths prior to the 1998 survey, and berm widths decline again over the following years as shown in the 2000 and 2002 surveys. This is particularly true at the northern end of the project; at the south end of Bal Harbour some accretion temporarily occurs as noted by increasing berm widths at R-30 and R-31 in the 2000 survey, a probable result of the southerly transport of material. Another trend uniformly observed in this portion of table 6 is the formation of a stable pocket beach in the vicinity R-27, at the north end of Bal Harbour. Material is impounded against the south jetty in a filet formation; aerial photographs and survey data suggest that this pocket beach remains in place along the northernmost 500 feet +/- of the Bal Harbour shoreline even when the remaining reaches of the shoreline to the south are badly eroded.

The second data set (center of table 6) provides the changes in mhw positions between successive surveys. These values are the differences between the measurements presented in the first section of table 6, as described above. Due to the different time intervals between some of the surveys, an annualized rate of shoreline change has been calculated and is included in the table as shown. The cumulative shoreline changes measured from all post-construction county-wide surveys (between 1990 and 2002) are provided on the right side of this section of the table. The average shoreline position change for all 74 profiles along the length of the county is provided at the bottom of the table for each survey interval. The average cumulative mhw position change over this 12.4-year interval (June 1990 through November 2002) was –3.9 feet (average of all 74 profiles), or an annualized change of –0.3 ft/yr. The total shoreline position changes from this section of table 6 were plotted into the charts shown in figures 10a and 10b for easier visualization.

Figure 10a shows a graphic representation of actual mhw shoreline position changes which have been observed during each survey interval along the length of the BEC & HP project since initial project construction was completed. Figure 10b shows the average annual mhw change rate, expressed in values of ft/yr for the period 1990 to 2002 along the entire length of the Federal project. Both figures 10a and 10b include the effects of the mhw displacements due to the various project renourishments which occurred between the respective surveys. An examination of maintenance records shows that 16 beach fill events occurred along the length of the project during this 12.4-year monitoring period, resulting in larger apparent advances of the mhw line than would have occurred without the fill placements.

These apparent advances of shoreline position have been removed in the third (right-hand) section of table 6. The average values of mhw advances created by each of the 16 beach fill placements were calculated or measured; these values were then subtracted from the rates of mhw change calculated in the central portion of the table. It is readily seen that much larger rates of shoreline recession are observed when the effects of beach renourishment are removed from the database. The average mhw position change over this 12.4-year interval with the effects of beach fill removed was –124.8 feet (average of all 74 profiles), or an annualized change of –10.1 ft/yr. These adjusted rates reflect the mhw shoreline change which would have occurred without periodic beach renourishment, and approximate the existing ‘natural’ rate of shoreline change. These mhw change rates have been plotted into the graphs shown in figure 11a and figure 11b for easier visualization. These graphs are similar to those in figures 10a and 10b, but with the effects of beach fills removed.

The largest values of shoreline recession observed in this section of table 6 (and figures 11a and 11b) predictably coincide with areas of historically high erosional losses. Many of these areas have been renourished repeatedly since project completion, and have become known as erosional ‘hotspots’. The largest shoreline recessions are noted in the table and figures at several of these ‘hotspots’ : northern Sunny Isles (R7-9), Bal Harbour (R27-31), and three areas in Miami Beach : 63rd Street (centered on R46), 32nd Street (centered on R59), and the south end of the project (R73-74).

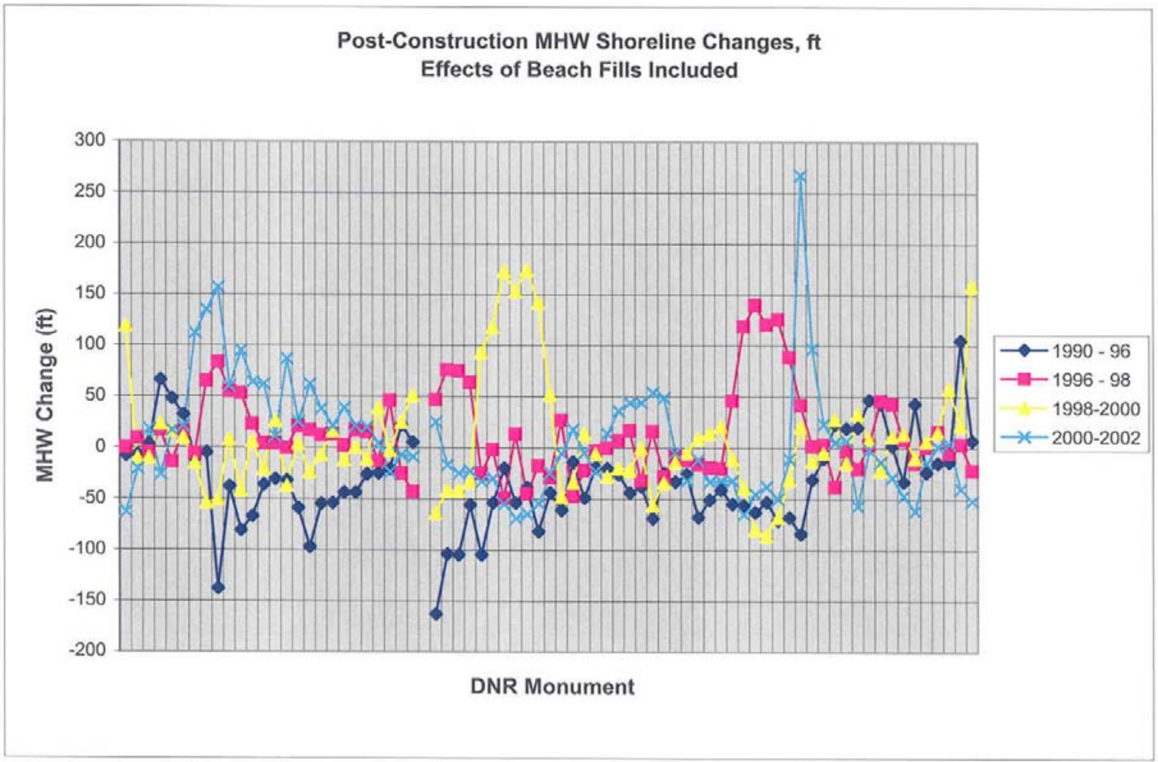


Figure 10a. Post-construction mhw changes (beach fills included)

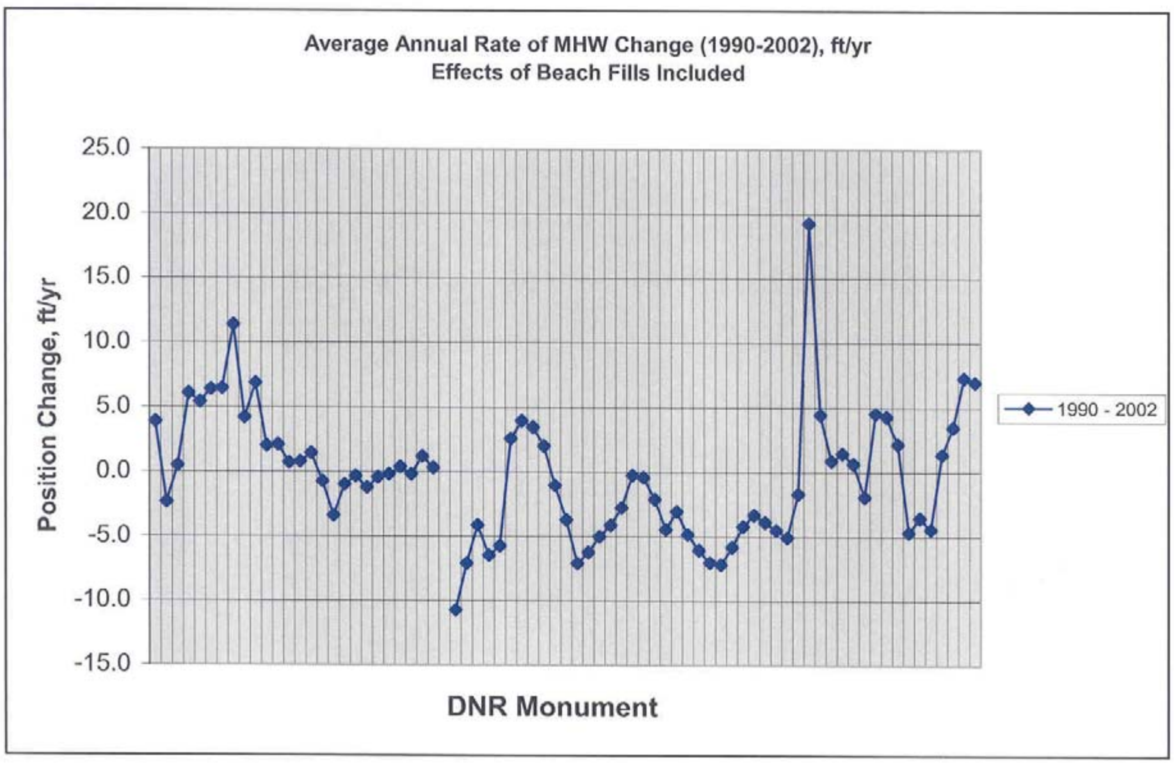


Figure 10b. Post-construction mhw changes (beach fills included)

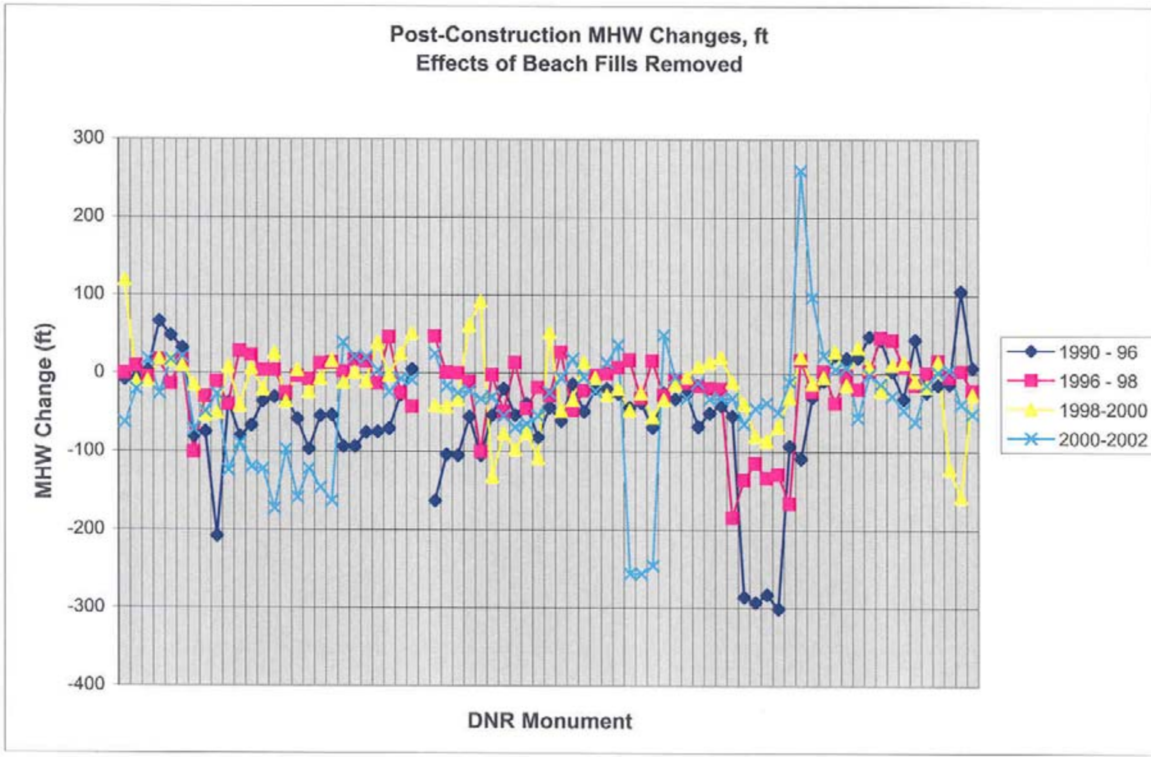


Figure 11a. Post-construction mhw changes (beach fills removed)

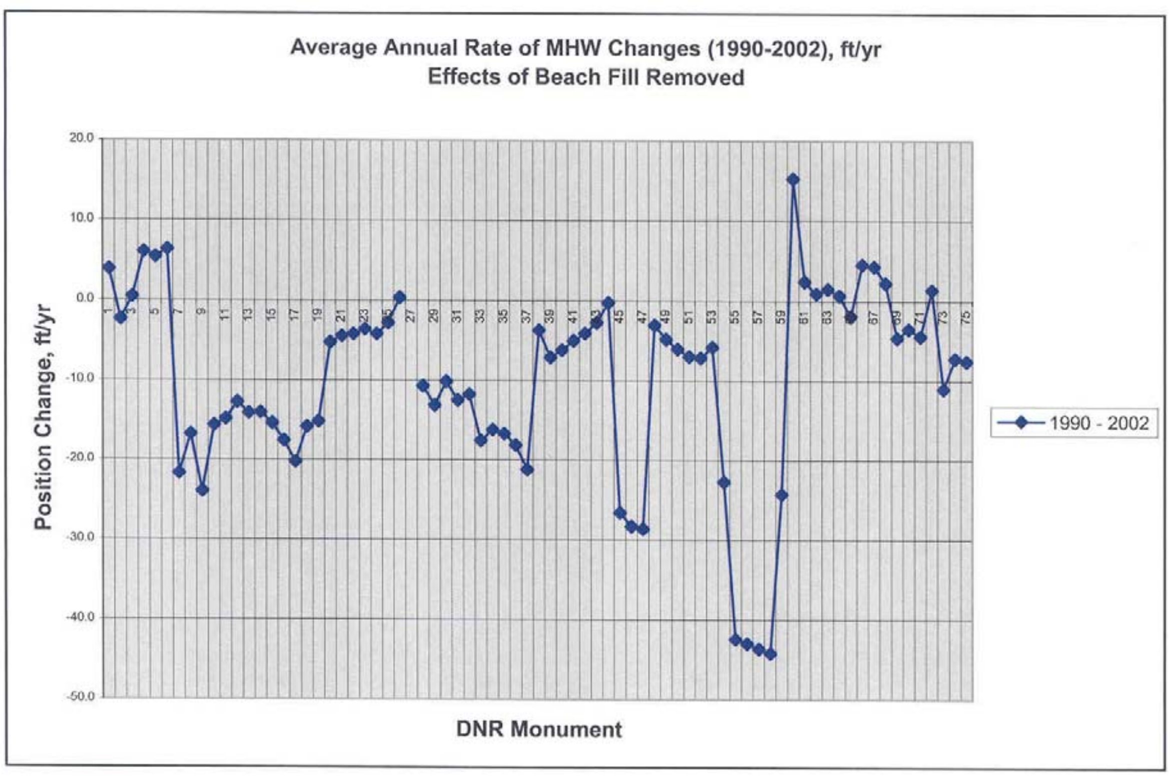


Figure 11b. Post-construction mhw changes (beach fills removed)

The highest losses occurred along a reach of Miami Beach near 32nd Street between monuments R53-R59, where high recession rates are observed during each survey period, and cumulative shoreline recessions of over 500 feet were measured over this 12.4-year post-construction survey period. Additional erosion control measures have been constructed at several of these hotspots, and are planned for construction at others. Breakwaters have been constructed at the erosional hotspots at northern Sunny Isles (in 2001) and 32nd Street (2002), and are planned for the 63rd Street hotspot (2005). The north jetty at Government Cut was sand-tightened in 1999 to eliminate losses from that hotspot, and the one remaining area of high erosional losses is Bal Harbour, the subject of this report.

In order to put the shoreline change rates from table 6 into perspective, average annualized shoreline change rates for each of the coastal communities of Dade County were calculated, to allow comparisons of performance of various segments of the Dade County shoreline. From the data contained in table 6, a summary of mhw changes is presented by coastal communities in table 7 below. Two columns of data are provided; the left column includes beach fill effects, in the right column the effects of beach fill placements have been removed.

Table 7

Average MHW Shoreline Change for
Dade County Coastal Communities

	With Beach Fill Effects (ft/yr)	Beach Fill Effects Removed (ft/yr)
Golden Beach	3.3	3.3
Sunny Isles	2.4	-16.8
Haulover Park	0	-3.5
Bal Harbour	-6.8	-11.7
Surfside	2.2	-18
Miami Beach (north)	-4.2	-16.7
Miami Beach (south)	2.6	-0.5
Countywide Average:	-0.3	-10.1

Again it is seen that recession of the mhw line is uniformly greater with the effects of beach fills removed, with the exception of the town of Golden Beach, which lies north of the Federal project and has received no beach fills during this monitoring period. The Miami Beach segment of the shoreline is broken into northern and southern segments for this analysis because the northern half is generally erosive, while the southern half is accretionary. From table 7 it is seen that Bal Harbour shows the highest mhw recession rate when beach fill effects are included, possibly a result of the 1990 beach fill placement artificially widening the beach immediately before the monitoring period began, and due to high losses from the 1998 renourishment. The county-wide average mhw change rate is -0.30 feet per year, demonstrating the stabilizing effects of beach fill placements.

When the effects of beach fill placements are removed the shoreline recession rates predictably increase. From the right-hand column of table 7 it is seen that Bal Harbour's average recession rate with the effects of beach fills removed is –11.7 feet per year, slightly higher that the county-wide loss rate of –10.1 feet per year. Recession rates are higher than Bal Harbour's in three other communities : Sunny Isles, Surfside, and northern Miami Beach, but the results in table 7 do not reflect the reduced losses due to the recent construction of stabilizing structures at Sunny Isles and Miami Beach.

When these post-project rates are compared to the pre-project rates calculated in the previous section of this report it is noted that recession rates are generally much higher in the post-construction era along the entire county. For the Bal Harbour region, the average annual shoreline change rate from 1867-1961 was +1.1 ft/yr (with a high degree of variability), compared to the post-project rate of –6.8 ft/yr (-11.7 ft/yr without beach fill effects). As cautioned earlier, the pre-project rate may include the effects of infilling and coastal armoring from construction along the shoreline during the early 1900's, and may therefore be artificially inflated. Regardless, the difference in shoreline recession rates before and after construction of the BEC & HP project is significant. One cause for the increased recession rates may be the interruption of littoral transport caused by the jetty/channel system at Bakers Haulover Inlet. Another factor contributing to the increased losses may be the accelerated erosion which typically results as artificially-placed beach fills undergo slope adjustment and end losses in response to the wave environment. These phenomena will be discussed in greater detail in following sections of the report.

<u>Volumetric Change Analysis.</u>

This analysis follows the same methodology as the mhw shoreline position change analysis presented above. Separate volumetric analyses will be conducted for the time period prior to construction of the Federal project, and for the period following completion of construction of the Federal project. The same set of pre- and post- construction surveys used in the mhw analysis will be used in this volumetric analysis.

Pre-Project Volumetric Changes. The surveys taken between 1883 and 1961 were used in the 1975 Dade County GDM (reference 1b) as a basis for determining volumetric changes prior to construction of the Federal project. Not all of these historical surveys were of adequate scope to allow volume computations to be performed. Table 8 is a summary of volumetric changes calculated in the 1975 GDM, based on selected historical surveys along the reach of shoreline between Government Cut and Bakers Haulover Inlet, and extending 3,900 feet north of Bakers Haulover Inlet through most of the length of Haulover Park. The older series of survey monuments is used in this table, as described previously.

Table 8
Pre-Project Volumetric Changes (1883-1961)

Change — Total Profile (1,000 cubic yards)

Government Cut to Bakers Haulover Inlet

Profile	Total 1883-1919 Accretion	Total 1883-1919 Erosion	Total 1883-1919 bracketed total	Average Annual[1] Accretion	Average Annual[1] Erosion	Average Annual bracketed total
Jetty – 1a	445	--		30	--	
1a – 2	1,170	--		78	--	
2 – 3	828	--		55	--	302[2]
3 – 4	820	--	4,533[2]	54	--	
4 – 5	676	--		45	--	
5 – 6	594	--		40	--	
6 – 7	--	167		--	11	
7 – 8	--	602		--	40	
8 – 9	--	258		--	17	81[2]
9 – 10	--	72		--	5	
10 – 11	--	111	1,215[2]	--	8	
11 – 12	217	--		14	--	
12 – 13	253	--		17	--	
13 – 14	46	--		3	--	
14 – 15	--	5	1,643[2]	--	0	109[2]
15 – 16	408	--		27	--	
16 – 17	445	--		30	--	
17 – 18	126	--		8	--	
18 – 19	148	-		10	--	

Change — Landward of -15 ft., m.l.w. (1,000 cubic yards)

3,900 ft. each side of Bakers Haulover Inlet

Profile	Total 1927-1961 Accretion	Total 1927-1961 Erosion	Total 1927-1961 bracketed total	Average Annual Accretion	Average Annual Erosion	Average Annual bracketed total
18 – 19	--	1,086		--	32	
19A – 19B	--	310		--	9	
19B – 19C	--	124	493[2]	--	4	15[2]
19C – 19D	--	59		--	2	

[1] The offshore bottom contours in 1883 are assumed to be equal to the offshore bottom contours in 1904.

[2] Total changes for bracketed reaches

The left side of table 8 shows total and average annual volumetric changes from the time period 1883 - 1919, extending from Government Cut at profile 1a northward to Bakers Haulover Inlet at profile 19. Again, profiles 18 and 19 lie within the limits of Bal Harbour. The right side of table 8 shows total and average annual changes during the period 1927 – 1961 along reaches of shoreline extending 3,900 feet to each side of Bakers Haulover Inlet. It is unclear why only these time intervals were analyzed in the 1975 GDM and why the analysis does not include the northern reaches of the county, but no records of the original surveys exist in the Jacksonville District; only this summary table is available.

In the analysis presented in the GDM it was assumed that the conditions from the 1883 survey were nearly identical to conditions in 1904 when construction on the Government Cut north jetty began. The time interval of 15 years was therefore used in computing the historical average annual volumetric changes along the reach of shoreline from Government Cut to Bakers Haulover Inlet. This 9.3-mile reach of shoreline was broken into three sections in this analysis. The southernmost section extended 15,000 feet north of Government Cut, and showed an average annual accretion of 302,000 cubic yards per year, which equates to +20.1 cy/lf/yr. The next section extended 13,000 feet farther north along the shoreline, and indicated an average annual erosion of –81,000 cubic yards per year, or –6.2 cy/lf/yr. The northernmost section extended along the remaining 21,000-foot reach to Bakers Haulover Inlet, and accreted at an average annual rate of 109,000 cubic yards per year, or +5.2 cy/lf/yr. It should be noted that no jetties or dredged navigation channel existed at Bakers Haulover Inlet during this time period.

Post-Construction Volumetric Changes. Volumetric analyses were performed for the post-BEC&HP construction period using the county-wide surveys conducted in 1990, 1996, 1998, 2000, and 2002. The year 1990 was chosen as the beginning point of this analysis because all project segments had been constructed at that time and the Federal project was in its present-day configuration. The area of coverage for these volumetric computations extends from the ECL seaward to about 1,200 feet. This offshore distance coincides with the location of the nearshore reef along the northern reach and some portions of the central reach of the Dade County shoreline, and an examination of profiles data shows that there is a high degree of closure at this depth along the entire length of the county. The landward limit of the volumetric analysis extends along the seaward edge of the dune (or seawall, as the case may be). A digital terrain model (dtm) was created for each of the five surveys, and volumetric changes were calculated between adjacent DNR profile lines using CADD survey analysis software. The volumetric changes obtained from this analysis are summarized in table 9.

Values in table 9 represent volume changes in units of cubic yards per linear foot of shoreline per year (cy/lf/yr) along the entire length of the Federal BEC & HP project. These units were used in order to eliminate the effects of the differing distances between profiles, and differences in time intervals between surveys. The use of these units allows easy comparison of volumetric changes along the length of the project, and with the historical database. Positive values represent accretion, and negative values represent erosion.

TABLE 9
POST-BEC PROJECT VOLUMETRIC CHANGE ANALYSIS
Volumes in units of CY/LF/YR

R #	Renourishments Included 1990-96	1996-98	1998-2000	2000-2002	1990-2002	Renourishments Factored Out 1990-96	1996-98	1998-2000	2000-2002	1990-2002	
											N. County Line
1	-0.2	0.8	-6.6	7.9	0.5	-0.2	0.8	-6.6	7.9	0.5	
2	6.1	-3.0	-1.6	3.5	2.9	6.1	-3.0	-1.6	3.5	2.9	
3	14.6	-6.4	16.6	1.7	9.0	14.6	-6.4	16.6	1.7	9.0	Golden Beach
4	14.6	-4.3	-0.4	9.1	8.1	14.6	-4.3	-0.4	9.1	8.1	
5	11.5	-0.5	-10.4	13.7	6.5	11.5	-0.5	-10.4	13.4	6.4	
6	10.1	-6.8	5.1	20.5	8.6	10.1	-6.8	5.1	19.8	8.4	
7	7.7	-0.8	-17.4	38.4	8.2	3.6	-16.1	-17.4	15.5	-0.7	
8	9.3	20.0	-21.6	31.1	10.3	7.3	7.5	-21.6	8.2	2.8	
9	9.0	17.6	-9.0	31.7	11.9	9.0	4.7	-9.0	8.8	5.4	
10	4.5	7.1	-17.3	37.7	7.8	4.5	6.6	-17.3	14.8	3.3	
11	-9.7	6.7	-13.8	31.4	0.2	-9.7	6.7	-13.8	8.5	-4.2	
12	-34.0	-0.5	-7.8	25.6	-12.8	-34.0	-0.5	-7.8	2.7	-17.3	
13	-15.8	-5.4	7.6	11.9	-5.0	-15.8	-5.4	7.6	-11.0	-9.4	Sunny Isles
14	4.9	-1.4	-10.4	15.2	3.4	4.9	-1.4	-10.4	-7.7	-1.0	
15	2.9	0.5	-7.7	17.2	3.6	2.9	0.1	-7.7	-5.7	-0.9	
16	-1.3	2.0	-7.8	21.6	2.6	-1.3	1.6	-7.8	-1.3	-1.9	
17	-5.4	0.9	-9.9	18.3	-0.5	-5.4	0.9	-9.9	-4.6	-4.9	
18	-4.8	0.9	1.6	4.6	-1.0	-4.8	0.9	1.6	-18.3	-5.4	
19	-3.3	4.1	-1.9	12.4	1.1	-3.3	4.1	-1.9	12.4	1.1	
20	-2.0	3.2	-2.2	7.0	0.5	-2.8	3.2	-2.2	7.0	0.1	
21	-1.9	5.6	-15.4	13.4	0.1	-2.7	5.6	-15.4	13.4	-0.3	
22	-3.7	4.1	4.8	2.3	0.1	-4.5	4.1	4.8	2.3	-0.3	
23	-4.4	10.5	-0.6	10.0	1.4	-5.2	10.5	-0.6	10.0	1.0	Haulover Park
24	-1.8	9.6	-8.1	3.2	0.0	-2.6	9.6	-8.1	3.2	-0.4	
25	-0.5	8.1	4.6	4.1	2.6	-0.5	8.1	4.6	4.1	2.6	
26	-4.6	-1.2	-30.8	-4.9	-8.3	-4.6	-1.2	-30.8	-4.9	-8.3	
26.5											
											Bakers Haulover Inlet
27	-12.1	13.7	-25.8	1.1	-7.6	-12.1	13.7	-25.8	1.1	-7.6	
28	-13.0	13.2	-10.8	-8.9	-7.6	-13.0	-31.8	-10.8	-8.9	-14.9	
29	-13.1	6.4	13.3	-9.8	-5.0	-13.1	-38.6	13.3	-9.8	-12.3	Bal Harbour
30	-12.4	-9.9	23.1	-8.8	-5.6	-12.4	-54.9	23.1	-8.8	-12.8	
31	-7.6	-14.2	37.1	-7.1	-1.3	-7.6	-14.2	6.8	-7.1	-6.2	
32	-3.9	-16.2	47.8	-4.3	2.4	-3.9	-16.2	-12.9	-4.3	-7.4	
33	-5.0	-8.8	52.9	-6.9	3.4	-5.0	-8.8	-7.8	-6.9	-6.4	
34	-6.9	-3.0	44.1	-7.7	1.8	-6.9	-3.0	-16.6	-7.7	-8.0	Surfside
35	-8.9	-7.7	39.6	-13.1	-1.7	-8.9	-7.7	-21.0	-13.1	-11.5	
36	-7.9	-5.8	23.0	-7.6	-2.5	-7.9	-5.8	23.0	-7.6	-2.5	
37	-6.0	-3.8	-0.7	3.8	-2.9	-6.0	-3.8	-0.7	3.8	-2.9	
38	-5.5	-7.7	-16.9	10.0	-4.7	-5.5	-7.7	-16.9	10.0	-4.7	
39	-3.7	-16.9	6.2	1.7	-3.2	-3.7	-16.9	6.2	1.7	-3.2	
40	-1.4	-14.7	-6.4	8.6	-2.4	-1.4	-14.7	-6.4	8.6	-2.4	
41	0.3	-5.3	-10.3	11.4	-0.2	0.3	-5.3	-10.3	11.4	-0.2	
42	-3.9	-2.5	-9.0	13.1	-1.2	-3.9	-2.5	-9.0	13.1	-1.2	
43	-7.9	5.9	-17.4	13.9	-3.0	-7.9	5.9	-17.4	13.9	-3.0	
44	-7.9	2.0	-4.1	13.9	-1.4	-7.9	2.0	-13.4	-14.3	-8.4	
45	-11.1	6.2	-18.2	16.6	-4.1	-11.1	6.2	-18.2	-11.6	-9.6	
46	-8.1	0.8	-14.7	12.8	-3.7	-8.1	0.8	-14.7	-1.3	-6.4	
47	-4.5	-12.4	-12.5	11.5	-4.0	-4.5	-12.4	-12.5	11.5	-4.0	
48	-5.4	-4.5	-17.0	5.2	-5.1	-5.4	-4.5	-17.0	5.2	-5.1	
49	-11.1	-2.4	-25.2	9.5	-8.0	-11.1	-2.4	-25.2	9.5	-8.0	
50	-10.4	-6.2	1.2	-7.5	-7.3	-10.4	-6.2	1.2	-7.5	-7.3	
51	-6.6	-5.2	-2.0	-2.5	-4.8	-6.6	-5.2	-2.0	-2.5	-4.8	
52	-6.5	5.9	-4.7	-1.6	-3.2	-6.5	5.9	-4.7	-1.6	-3.2	Miami Beach
53	-11.0	30.4	-21.5	0.1	-3.9	-11.0	-12.5	-21.5	0.1	-10.8	
54	-8.5	30.5	-3.1	-3.2	-0.3	-11.6	-20.3	-3.1	-3.2	-10.0	
55	-9.6	34.4	-25.3	-9.1	-4.9	-24.5	-17.1	-25.3	-9.1	-20.5	
56	-11.1	35.6	-14.4	-9.5	-3.8	-14.4	-7.8	-14.4	-9.5	-12.4	
57	-10.9	24.8	-22.7	-2.2	-5.4	-11.9	-31.2	-22.7	-2.2	-14.8	
58	-8.5	4.2	2.1	14.9	-0.2	-9.5	-7.7	2.1	14.9	-2.6	
59	-7.3	-3.5	-5.1	36.5	2.2	-7.3	-2.9	-5.1	36.5	2.3	
60	-4.1	0.7	1.5	8.1	0.0	-4.1	0.7	1.5	8.1	0.0	
61	-0.4	-0.4	-8.3	11.1	0.5	-0.4	-0.4	-8.3	11.1	0.5	
62	1.3	-5.9	-13.7	14.2	0.2	1.3	-5.9	-13.7	14.2	0.2	
63	0.9	-7.5	3.2	-0.4	-0.3	0.9	-7.5	3.2	-0.4	-0.3	
64	2.9	-2.0	3.1	-6.1	0.4	2.9	-2.0	3.1	-6.1	0.4	
65	5.7	14.5	-8.5	4.5	4.6	5.7	14.5	-8.5	4.5	4.6	
66	5.1	10.6	-8.1	4.9	3.8	5.1	10.6	-8.1	4.9	3.8	
67	5.7	15.8	3.5	0.9	6.1	5.7	15.8	3.5	0.9	6.1	
68	2.9	6.6	-1.9	3.5	2.8	2.9	6.6	-1.9	3.5	2.8	
69	3.3	2.8	4.2	6.1	3.9	3.3	2.8	4.2	6.1	3.9	
70	-2.0	12.1	-2.3	14.6	3.4	-2.0	12.1	-2.3	14.6	3.4	
71	-5.7	7.5	-11.0	19.7	0.5	-5.7	7.5	-11.0	19.7	0.5	
72	-5.8	5.8	-1.0	9.4	-0.2	-5.8	5.8	-21.6	9.4	-3.5	
73	-5.0	3.8	26.2	-1.9	2.1	-5.0	3.8	-14.9	-1.9	-4.6	
74											Government Cut (Miami Hbr)

Table 9 is divided into 2 sections to allow for the effects of beach fill placement to be demonstrated. The left side of table 9 contains the raw volumetric calculations between consecutive surveys, and the effects of beach fill placements are included in these values. Sixteen beach fill placements were completed along the project between 1990 and 2002, corresponding to the time for which county-wide surveys are available. These beach fill placements are summarized below in table 10. The volumetric changes presented in the left side of table 9 include the effects of these added beach fill volumes, and as a result shows higher rates of accretion (and lower rates of erosion) along the reaches of shoreline where beach fill was placed. This data reflects the *measured* erosional losses along the length of the project, but does not depict the *total* erosional losses because of the effects of beach fill placement. For example, if a designated reach of shoreline eroded at a rate of 100,000 cy/yr, but was renourished with 100,000 cubic yards of fill between annual surveys, an analysis of these surveys would lead one to conclude that the *measured* erosion rate was zero, but by removing the volume of beach fill placement, the correct *total* erosion rate of –100,000 cy/yr is calculated.

TABLE 10
Dade County Beach Fill Placements (1990 – 2002)

Year	Volume, cy	Location	
1990	32,000	R-7 – R-9	Sunny Isles
1994	24,500	R-20 – R-25	Haulover Park
1994	122,000	R-54 – R-57	Miami Beach
1994	30,000	R-54 – R-59	Miami Beach
1996	8,000	R-54 – R-60	Miami Beach
1997	9,000	R-7 – R-8; R-10; R-16	Sunny Isles
1997	80,000	R-7 – R-10	Sunny Isles
1997	50,000	R-57 – R-59	Miami Beach
1997	35,000	R-54 – R-56	Miami Beach
1997	478,000	R-53 – R-58	Miami Beach
1998	282,000	R-28 – R-31	Bal Harbour
1998	18,000	R-44 – R-45	Miami Beach
1999	590,000	R-32 – R-36	Surfside
1999	132,000	R-73 – R-74	So. Miami Bch.
2001	704,000	R-7 – R-19	Sunny Isles
2001	167,000	R-44 – R46.5	Miami Beach

In order to remove the effects of these beach fill placements, the volumes of each of these 16 renourishments were removed from the data on the left side of table 9, along the respective areas of each beach fill project. The corrected volumetric changes are provided on the right side of table 9. By removing the volumes of each of the 16 fill placements along the reaches of affected shoreline, the 'without-project' erosion rates along the length of the project can be determined. Both data sets are included in this table because each shows an important aspect of project performance : the left side of the table shows the actual volumetric changes which have occurred since the Federal project was constructed; the right side approximates the volumetric changes which would have occurred if the Federal project had not been constructed. The values on the right side of table 9 are especially significant because they more closely depict the "natural" erosion rates along the length of the project, and more accurately represent the erosion potential along the length of the Federal project in its present configuration.

The left side of table 9 shows volumetric changes measured between adjacent DNR monuments along the length of the project, to an offshore distance of 1,200 feet. Four survey intervals are depicted : 1990-96, 1996-98, 1998-2000, and 2000-2002. A summary of cumulative volume changes from 1990 – 2002 is shown in the right-hand column on each side of the table. As seen in table 9, no clear large-scale trends toward erosion or accretion are observed along the left side of the table, primarily because several erosive areas within the project were renourished (some repeatedly) during these time intervals. Although values of erosion and/or accretion can be quite high along some reaches of the project, it is seen in the left side of the table that values of erosion during one survey interval are usually balanced by accretion during the next interval as natural recovery occurs, and as beach renourishment projects are constructed to repair erosional damage to the project. The net result is that the shoreline is relatively stable, and overall long-term volumetric changes are comparatively low.

For example, one of the most historically erosive areas of the project lies between monuments R-50 and R-60. From the left-side of table 9, it is seen that each profile in this area experienced a moderate degree of erosion between 1990 and 1996 (in spite of two beach fill placements in 1994 totaling 152,000 cy between R-54 and R-59). A large amount of accretion occurred during the two-year interval from 1996-98, the result of a large beach renourishment project (486,000 cy placed between R-53 and R-60 during this time). During the interval 1998-2000 this area experienced erosion at a higher rate than before renourishment, as the recently-placed beach fill experienced rapid losses during stabilization (and no further beach fill placements were made during this time). The fill appears to have stabilized from 2000-2002, at least partially due to the construction of stabilizing headland structures near R-59. As seen in the right-hand column of this section of table 9, in spite of the large fluctuations of erosion and accretion between R-50 and R-60 during this 10-year period, the net unit volume changes between 1990 and 2002 were very low, as beach fill placements nearly balanced erosional losses.

The remaining portions of the Dade County shoreline behaved in a similar fashion : relatively large fluctuations in unit erosion/accretion rates occurred during individual survey periods, but much smaller shoreline changes were observed when averaged over the full 12.4-year time period. The project as a whole demonstrates an overall tendency to erode along most of the project's length, and much of the accretion observed in the left side of table 9 is the result of beach fill placements. The summary of net volumetric changes from 1990 to 2002 along the Dade County shoreline is shown in the right-hand column of this section of table 9. This data shows that north of Bakers Haulover Inlet the shoreline generally accreted at low to moderate rates during this period, with some limited areas of minor erosion, primarily in central Sunny Isles and the south end of Haulover Park. From Bakers Haulover Inlet to Government Cut the shoreline eroded at low to moderate rates over the entire reach during the same period, the only exceptions being along northern Surfside and along 'South Beach' (R-63 – R70), a historically accretionary region of southern Miami Beach.

For easier visualization, data for cumulative volumetric changes during the period 1990 – 2002 from both sides of table 9 were plotted into the bar charts shown in figures 12a and 12b. Each of these graphs represents the length of the Dade County shoreline extending from DNR-1 at the top to DNR-74 at the bottom. The locations of each of the coastal communities are indicated on the right-hand margin of each graph. Both graphs present volumetric changes in units of cy/lf/yr in order to remove the effects of differing profile line spacings and different survey intervals. Figure 12a includes the effects of the sixteen beach fill placements during this period, figure 12b removes the effects of fill placements.

As seen in table 9 and figure 12, the northern portion of the project from the Dade County line to Bakers Haulover Inlet (R-1 through R-26), generally remains more stable than the southern portion of the project (R27 through R-74). With few exceptions, erosion rates along the northern reach of the project are lower for each survey interval. From 1990 through 1996, most of this reach accreted, with slight erosion noted along Haulover Park. Between 1996 and 1998 this trend reversed somewhat, with a higher degree of erosion at the northern end of Sunny Isles, slight erosion through most of Golden Beach, and low to moderate accretion throughout most of the remainder of this reach. From 1998 to 2000 most of Sunny Isles eroded at moderate to occasionally high rates, with moderate levels of interspersed erosion and accretion along the remainder of this reach. During the interval 2000-2002 the effects of the large beach fill placement along Sunny Isles can be seen. Examination of the net volumetric changes from 1990 to 2002 contained in table 9 and figures 12a and 12b show that moderate accretion occurred along Golden Beach and northern Sunny Isles, while the southern portion of Sunny Isles and Haulover Park eroded at low to moderate levels. The large spike of erosion observed between R-12 and R-13 in figures 12a and 12b appears to be due to an anomaly in the 1996 survey; no corresponding severe erosion was observed in this area. Net rates of volumetric change along the shoreline north of Bakers Haulover Inlet were accretionary to mildly erosive, with most values less than 5 cy/lf/yr. Losses along the north county reach were reduced further by the completion of the Sunny Isles breakwater in 2001.

A more definite trend is observed along the southern reach of the county, extending from Bakers Haulover Inlet to Government Cut (R-27 through R-74). Erosion is noted along nearly this entire reach of the coast during each survey interval, with the only consistent accretion occurring along the "South Beach" portion of Miami Beach, in the vicinity of monuments R-63 through R-70. The most consistent rapidly-eroding areas, as seen in the right-hand section of table 9 and figure 12, are Miami Beach in the area of R-53 through R-58, and Bal Harbour (R-27 – R-31). Other areas which experienced higher than average erosion rates are Surfside (R-32 – R-36), and northern Miami Beach (vicinity of R-46).

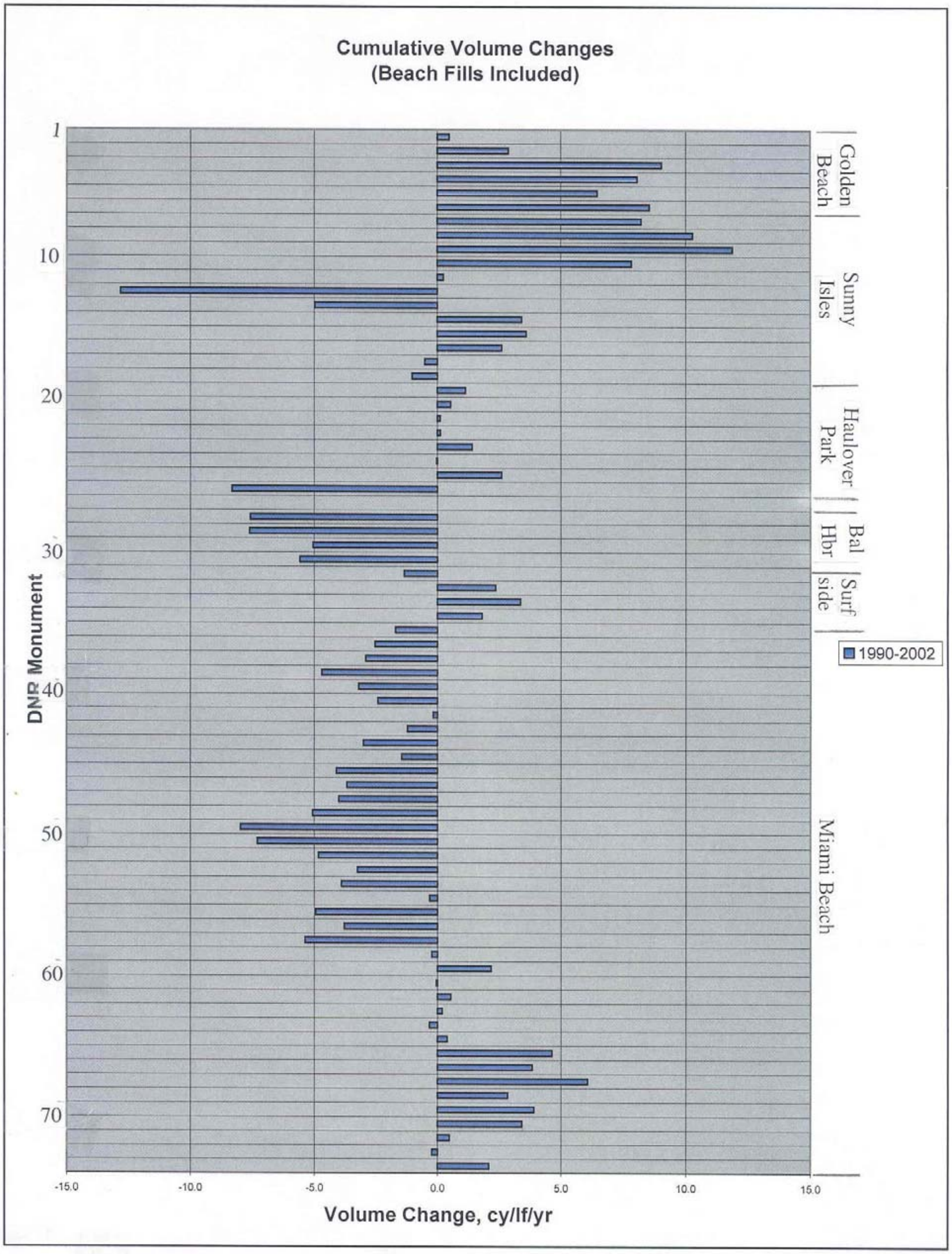


Figure 12a. Cumulative Volume Changes (1990-2002) with Beach Fills Included.

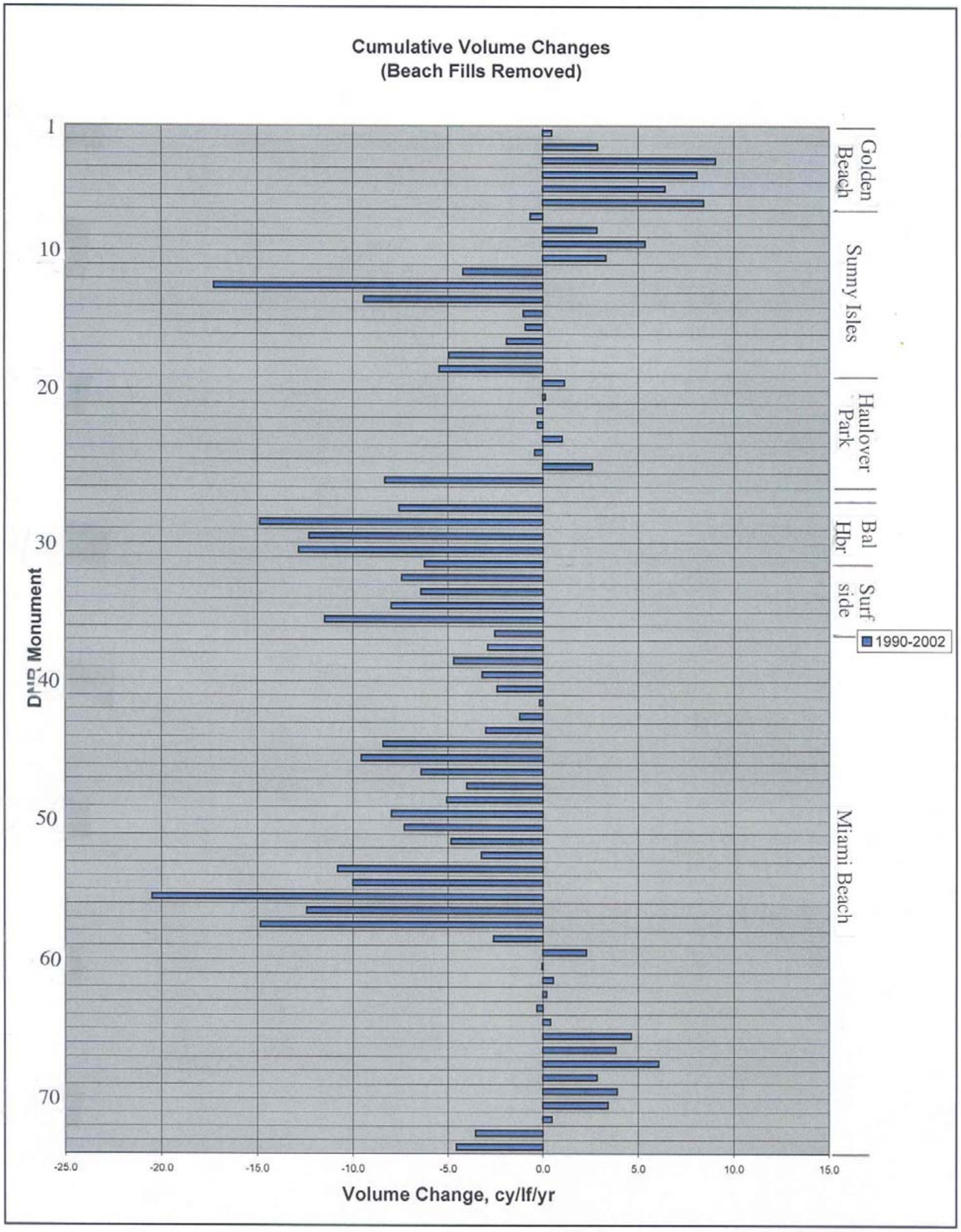


Figure 12b. Cumulative Volume Changes (1990-2002) with Beach Fills Removed.

As seen in table 9 and figure 12b, the portion of Miami Beach in the vicinity of R-53 through R-57 shows the highest long-term erosion rates along the Dade County shoreline. This persistent, rapid erosion is further evidenced by the large number of fills placed along this reach during this 12-year monitoring period. Six fills totaling 723,000 cubic yards were placed along this 1-mile reach of shoreline between 1990 and 2000. Cumulative erosion rates along the reach from R-53 to R-58 range from –10.0 to –20.5 cy/lf/yr, with an average rate of erosion through this reach of –13.7 cy/lf/yr.

The Bal Harbour shoreline had the second-highest rate of erosion along the Dade County shoreline during the 1990-2002 period, as shown in table 9 and figures 12a and 12b. Cumulative erosion rates varied from –6.2 to –14.9 cy/lf/yr between R-27 and R-31, with an average rate of erosion along this reach of –10.8 cy/lf/yr (removing fills) over the 12.4-year monitoring period. Even including the effects of beach fills an average unit erosion value of -5.4 cy/lf/yr is calculated. From the left side of table 9, erosion to the 1990 beach fill occurred throughout Bal Harbour during the interval from 1990-96. The accretion shown during the 1996-1998 interval reflects the volume of fill placed prior to the 1998 survey. From 1998-2000 this beach fill eroded along the northern half of Bal Harbour but accreted along southern Bal Harbour, probable evidence of the southward littoral transport of sediment in the area. Erosion occurred along most of Bal Harbour from 2000 – 2002, but at the relatively moderate rates of about –8 cy/lf/yr. The highest rates of erosion observed along Dade County during the 1990-2002 monitoring period occurred along central Bal Harbour between 1996-98 (-54.9 and –38.6 cy/lf/yr; beach fills removed). One beach renourishment project was constructed along the Bal Harbour shoreline during this monitoring period, consisting of the placement of 282,000 cubic yards of fill between R-28 and R-31 in 1998. Note that the most recent (June 2003) Bal Harbour renourishment was not included in this regional analysis due to the lack of a 2003 county-wide survey. Effects of that fill will be discussed in following sections of this report.

The Surfside shoreline also showed persistent erosion, although lower than that observed at Bal Harbour. Erosion occurred at each profile along the entire 1-mile length of the city with the exception of the 1998-2000 interval, when beach renourishment was performed. Unit rates of erosion were low to moderate throughout all survey intervals, even with the effects of beach fills removed. Cumulative volumetric change rates varied from +3.4 to -2.5 cy/lf/yr (-2.5 to –11.5 cy/lf/yr with fills removed), and the average 12-year erosion rate was +0.7 cy/lf/yr (–7.2 cy/lf/yr with fills removed). In 1999, one beach fill consisting of 590,000 cubic yards was placed along the entire length of Surfside, from monuments R-32 to R-36. The effects of this beach fill placement are seen in table 9 and in a comparison between figures 12a and 12b. Note from table 9 that accretion into southern Bal Harbour occurs during this same survey interval, probably due to the spreading (end losses) from this fill.

The area of Miami Beach near R-46 (63rd Street) also experienced relatively high erosion during this 12-year period. As seen in table 9, this erosive area is broad and not well defined. Several profiles along the area from R-43 to R-50 experienced moderate erosion, with areas of lower erosion to the north and south. The highest average loss rate observed along this reach was –8.0 cy/lf/yr; the lowest rate was –1.4 cy/lf/yr, with an average loss rate of –4.6 cy/lf/yr. In 1998, one beach fill consisting of 18,000 cubic yards was placed between monuments R-44 and R-45. Factoring out the effects of this fill gives an average loss rate of -6.5 cy/lf/yr.

Two areas of accretion are noted along the Dade County shoreline, at Golden Beach (R-1 through R-6), and along the "South Beach" portion of Miami Beach (R-63 through R-70). Cumulative gains of up to +9.0 and +6.1 cy/lf/yr were noted at these two areas, respectively. Golden Beach is located between two large Federal Beach Erosion Control projects, and end losses from these large fills sustain the Golden Beach shoreline. The South Beach region of Miami Beach is located along a large shallow embayment just north of the Government Cut north jetty. The southerly littoral transport of sediment naturally tends to accrete within the embayment, and against the north side of the north jetty. Both areas have historically been stable or accretionary, and no beach fills were placed along either reach of shoreline during the 1990-2000 monitoring period.

Analysis of survey data from 1990 through 2002 provides some indication of future erosion rates and renourishment needs, since all project components had been constructed (and had presumably reached equilibrium) by the beginning of this time period. It is seen from this volumetric analysis that the Dade County BEC & HP eroded a total of 3,887,144 cubic yards during this 12.4-year period, equating to an average annual erosion rate of – 313,480 cubic yards per year, between R-1 and R-74. This value was obtained by factoring out areas of accretion along the project. As has been discussed in this report, much of this erosion is confined to several erosional 'hotspots' along the length of the project, and additional shore protection features have been, or will soon be, constructed at several of these 'hotspot' areas. The overall erosion rate should decrease once these erosion control measures are in place.

Bakers Haulover Inlet.

Bakers Haulover Inlet lies immediately north of Bal Harbour, and due to its proximity to the study area maintenance dredging activities at the inlet can have an impact on the Bal Harbour shoreline. As previously discussed, Bakers Haulover Inlet is a Federally-authorized navigation project, and maintenance dredging of the inlet is coordinated with renourishment of the adjacent beaches (also Federal projects) to the maximum extent possible. Table 11 lists maintenance dredging events which have occurred since construction of the Federal BEC & HP project. These events are described in more detail under the "Project History" section of this report on pages 9 through 12.

Table 11

Maintenance Dredging Events (1980-2003) – Bakers Haulover Inlet

Year	Volume Dredged, cy	Area of Inlet Dredged	Disposal Area
1980	43,163	Flood shoal/IWW	Haulover Park
1984	35,000	Flood shoal/IWW	Haulover Park
1990	32,000	Flood shoal/IWW	N. Sunny Isles
1994	24,560	Flood shoal/IWW	Haulover Park
1998	282,852	IWW/Entrance channel	Bal Harbour
2003	188,000	Ebb shoal	Bal Harbour

These dredging events can impact the Bal Harbour shoreline in two primary ways : removal of material from the inlet generally results in a deepening of the entrance channel which in turn can further limit natural sediment bypassing around the inlet. Due to the predominant southward littoral drift, this can result in erosion along the Bal Harbour shoreline. On the other hand, material dredged from the inlet consists of beach-quality sand which is placed along the adjacent shorelines, offsetting erosion.

Secondary effects of inlet dredging are also possible. Dredging the entrance channel can result in increased tidal flow through the inlet. Increased tidal velocities (particularly during ebb flow) can result in a higher degree of refraction of incident waves, which may tend to focus wave energy along reaches of shoreline near the inlet. Similarly, excavation of the ebb shoal may also alter wave refraction patterns near the inlet, resulting in localized areas of wave energy focusing. Both of these processes will be examined in the wave refraction analysis section of this report using the numerical model STWAVE.

Longshore Transport Rates and Sediment Budget.

Numerous sediment budgets and longshore transport rates have been calculated for the Dade County coastline in several studies over the years. A summary of longshore transport rates from previous studies is provided in table 12. These values were calculated using several different methodologies, and for the most part are reasonably consistent in magnitude and direction of net sediment transport. It is seen that the more recent studies have presented longshore transport rates which are generally lower than those calculated during the initial planning and construction of the Federal project.

Table 12

Summary of Calculated Longshore Transport Rates

Net Transport Rate, Direction (cy/yr)	Study	Methodology Used
100-150,000 (s)	1958 UF Study	Shoaling analysis
< 50,000 (s)	1965 Feas. Study	Shoaling analysis
20,000 (s)	1975 GDM	Regional vol. changes,w/groins
217,000 (s)	1975 GDM	Regional vol. changes,w/o grns
235,000 (s)	1975 GDM	SSMO waves[1], SPM[2]
327,000 (s)	1982 Survey Report	WIS phase I waves, SPM[2]
95,000 (s)	1984 GDM Add II	WIS phase III waves, SPM[2]
50-150,000 (s)	1996 COF Report	Shoreline position changes
48-115,000 (s)	1997 CSI Sed. Budget	Regional volumetric analysis
117,000 (s)	2000 CSI 32nd St Analysis	Inman & Bagnold methodology

[1] SSMO = Surface Synoptic Meteorological Observations, ship-based wave database

[2] SPM = Shore Protection Manual longshore transport equations using energy flux method

A recent large-scale sediment budget was presented in the 1997 "Dade County Regional Sediment Budget" study performed for the local sponsor by CSI Inc. (reference 2d). The longshore transport rates presented in that analysis were based on volumetric changes along the shoreline and known boundary conditions at the north and south ends of the study area (Port Everglades and Government Cut, respectively). The CSI study provides the most comprehensive regional sediment budget available and will be used in this study.

Regional longshore transport rates were calculated along uninterrupted reaches of shoreline between inlets in the 1997 CSI regional sediment budget analysis. The Dade County shoreline falls within two such uninterrupted reaches – the north part of the county's shoreline falls within the beach segment extending from Port Everglades to Bakers Haulover Inlet, and the south part of the Dade County shoreline extends from Bakers Haulover Inlet to Government Cut. Longshore transport rates were calculated along each of these reaches based on volumetric change data from 1980 to 1996 in the CSI analysis. Figures 13a and 13b show longshore transport rates as well as net volumetric changes over these selected reaches of shoreline based on data presented in the 1997 regional sediment budget analysis.

The analysis begins at Port Everglades, where known boundary conditions exist. The jetties and navigation channel form a complete littoral barrier, and sediment transport rates south of the inlet can be calculated based on volumetric changes since no material enters the littoral system from the north. From the summary shown in figure 13a it is seen that the net transport past Port Everglades is zero, but generally increases to the south based on the volumetric changes indicated along the southern Broward County coast. At the Broward/ Dade county line the net southerly transport rate is 83,000 cy/yr. This value is used as the starting point for the updated sediment budget analysis presented in this report, since the surveys and analyses contained in this report do not extend into Broward County.

Beginning with the net southerly transport of 83,000 cy/yr at the northern county line, rates decrease to 60,000 cy/yr at the northern end of Sunny Isles, but increase to 64,000 cy/yr at the southern end of Sunny Isles, based on volumetric changes along the shoreline. At Bakers Haulover Inlet the net longshore transport has again declined to 60,000 cy/yr as seen in figure 13a. Note that a large degree of accretion occurs along Golden Beach, a result of in-filling from the large Federal beach renourishment projects located on each side of the Golden Beach city limits.

Figures 14a and 14b show sediment budgets at Bakers Haulover Inlet and Government Cut, respectively. These sediment budgets were developed by CSI, Inc. based on an analysis of the survey data used for development of the longshore transport rates, and on an examination of dredging records from the two inlets. As seen in figure 14a, of the 60,000 cy/yr entering Bakers Haulover Inlet from the north, 32,000 cy/yr is transported seaward into the ebb shoal, and 9,000 cy/yr is transported into the interior of the inlet including the flood shoal system. The entire 9,000 cy which is deposited along the interior of the inlet is mechanically placed back along the beaches, usually to the north of the inlet along Haulover Park. Therefore a total of 19,000 cy/yr bypasses the inlet naturally, in a southbound direction.

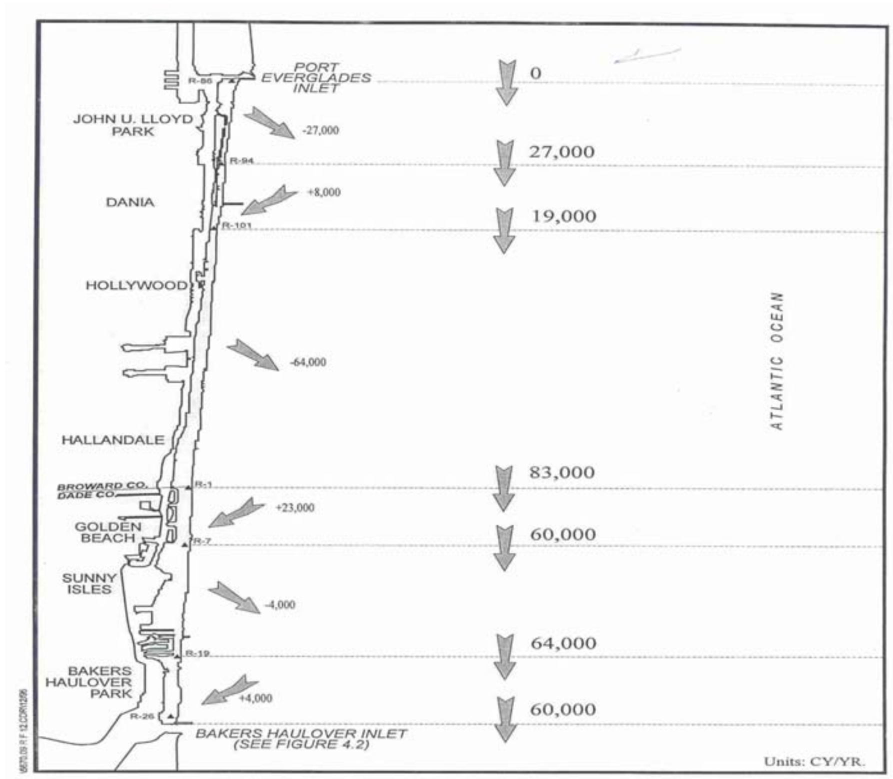


Figure 13a. Sediment Budget – Port Everglades to Bakers Haulover Inlet (CSI, 1997)

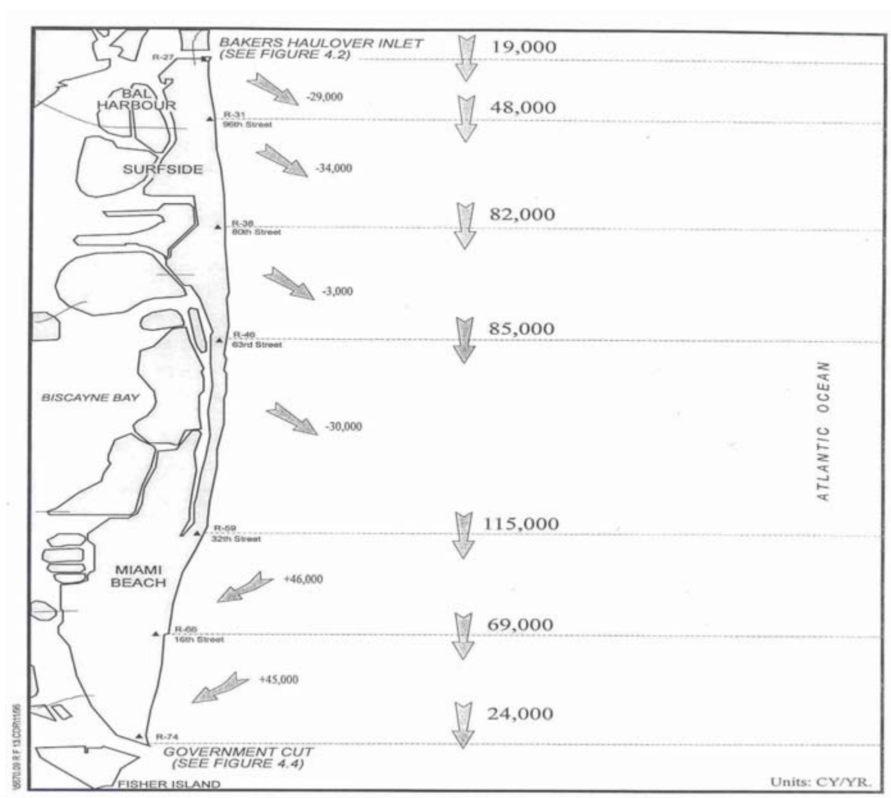


Figure 13b. Sediment Budget – Bakers Haulover Inlet to Government Cut (CSI, 1997)

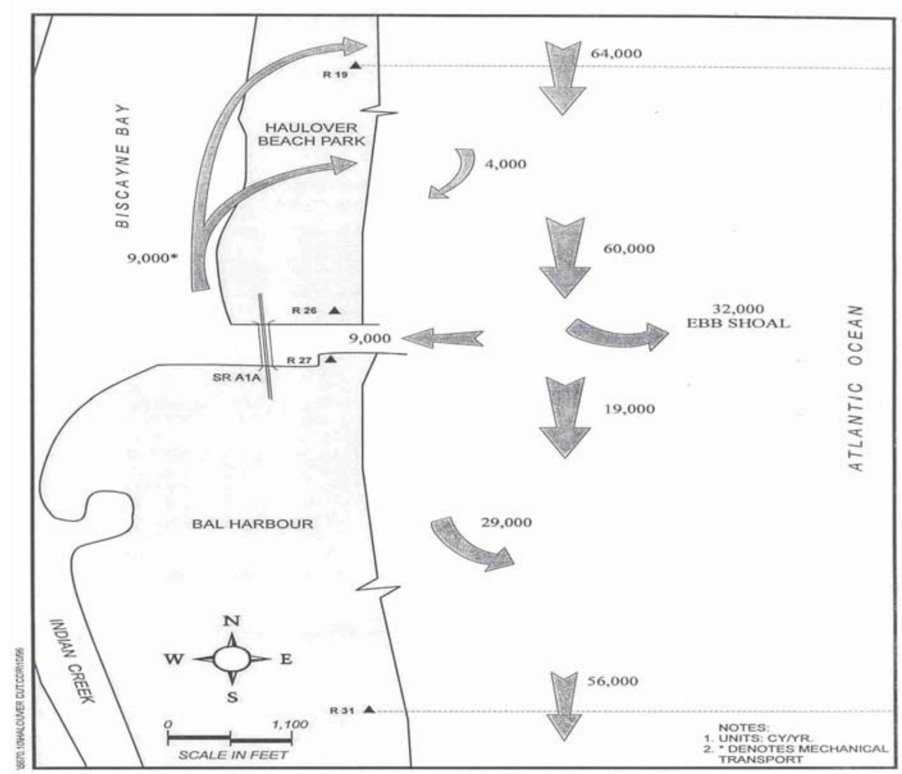


Figure 14a. Sediment Budget – Bakers Haulover Inlet (CSI, 1997)

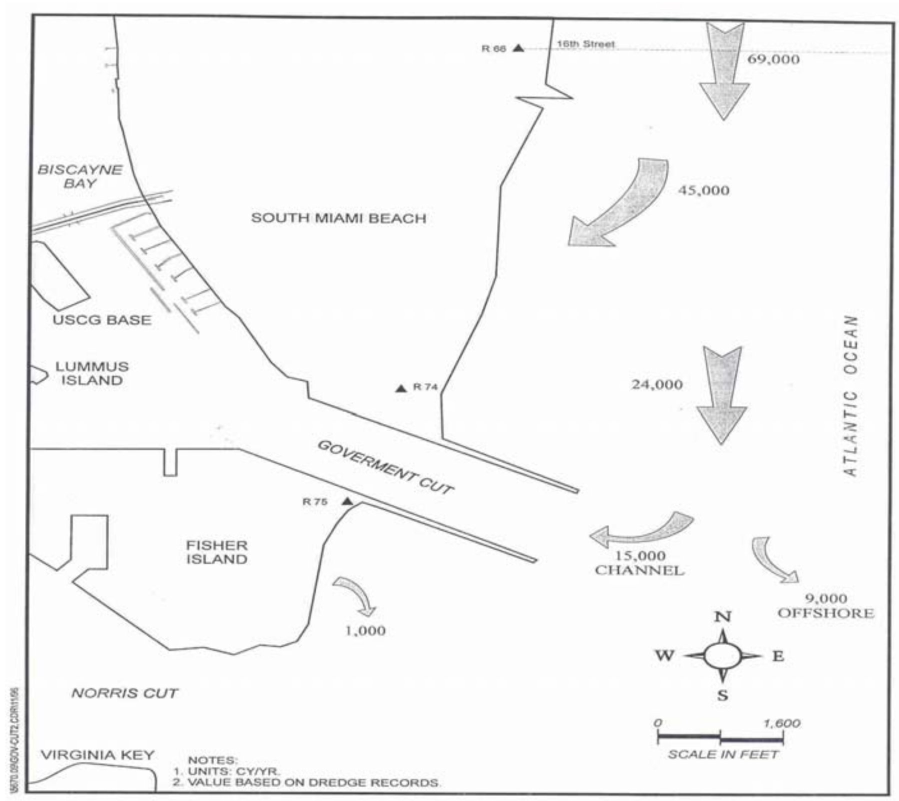


Figure 14b. Sediment Budget – Government Cut (CSI, 1997)

The net longshore transport rate at the north end of Bal Harbour is therefore 19,000 cy/yr, which represents material transported southbound along the ebb shoal. Survey data from 1980-96 shows that annual losses of 29,000 cy/yr occurred along Bal Harbour during that period, resulting in a computed net transport rate at the Bal Harbour/Surfside city limit of 48,000 cy/yr southbound, as shown in figure 13b. The primary pathway for sediment moving southbound around Bakers Haulover Inlet into Bal Harbour is the ebb tidal shoal at the inlet. Analysis of aerial photographs and recent lidar survey data show that the ebb shoal is a dominant bathymetric feature in the region, providing a partial "bridge" for sediment transport around the inlet (see lidar survey bathymetry, figure 18). Like most ebb shoal features, this shoal is roughly horseshoe-shaped, extending from the south-central portion of Haulover Park, several thousand feet seaward, and curving back to shore in the vicinity of north-central Bal Harbour (near monuments R28/29). Any proposed improvements to the Bal Harbour segment of the project should include an analysis of the effects on this important sediment pathway.

Annual losses of 34,000 cy/yr along Surfside result in a net southward transport rate of 82,000 cy/yr at the Surfside/Miami Beach city limit. Longshore transport rates along Miami Beach are calculated at two areas which have historically experienced high erosion rates – at 63rd Street and 32nd Street (R-46 and R-59, respectively). Annual losses of –3,000 cy/yr along the northern portion of Miami Beach result in a net transport rate of 85,000 cy/yr to the south at 63rd Street. The high degree of erosion (-30,000 cy/yr) noted along the reach between 63rd and 32nd Street accounts for the increase in the longshore transport rate in this region. The longshore transport rate at 63rd St is calculated to be 115,000 cy/yr southbound, the highest rate observed along the county's shoreline. This localized high transport rate is due primarily to waves striking this curved region of the shoreline at a higher angle than the surrounding areas.

Proceeding further southward, the net longshore transport rate begins to decrease by 16th Street (R-66) to 69,000 cy/yr in the CSI analysis. Due to the recurving of the coastline, incoming wave energy arrives at a more shore-normal angle, resulting in decreased longshore transport rates and high rates of accretion the remaining reaches of the project. As seen in figure 13b, the transport rate decreases to 24,000 cy/yr at the north side of Government Cut.

Figure 14b shows the sediment budget calculated for Government Cut, based on the longshore transport analysis presented above and an examination of dredging records from this Federal navigation channel. Government Cut, like Port Everglades, is considered to be a complete littoral barrier. The 24,000 cy/yr which enters the channel from the north is distributed as shown in figure 14b : 15,000 cy/yr is deposited in the interior of the inlet, and 9,000 cy/yr is deposited in shoals along the outer reaches of the channel. It is generally agreed that based on analyses of survey data, no material is transported southward past the 44-foot deep entrance channel.

SUMMARY OF PHYSICAL DATA – BAL HARBOUR STUDY AREA.

General.

The previous section of this report examined physical processes on a regional scale in order to provide an overview of the large-scale processes which affect the Bal Harbour study area. This section of the report will focus on the Bal Harbour study area, which is defined as extending through the length of Bal Harbour, and one mile to the north and to the south. The regional physical processes will be presented in greater detail, and with reference only to this study area and the problems being investigated in this report.

Project History.

The Bal Harbour shoreline is an authorized segment of the Dade County BEC & HP project, and was initially constructed in 1975 with the placement of 1.6 million cy between DNR monuments R-27 and R-31.5. The resulting berm extended 240 feet seaward from the ECL at an elevation of +9.0 feet, mlw. Three renourishments have been performed along Bal Harbour since initial construction. These projects were completed in 1990, 1998, and 2003, and are described in more detail in table 13. Each of these renourishments resulted in the reestablishment of the 240-foot construction berm width along the length of Bal Harbour.

Table 13
Bal Harbour Beach Fill Placements

Date	Volume Placed (cy)	Berm Width, ft	Comments
1975	1,600,000	240	Initial construction
1990	225,000	240	1st renourishment
1998	282,852	240	2nd renourishment
2003	188,000	240	3rd renourishment

In addition, several beach fills have been placed along the adjacent shorelines at Haulover Park (to the north) and Surfside (to the south). Beach fill placement in both of these adjacent areas have indirect effects on the Bal Harbour shoreline, but fill placed along Surfside provides more direct effects since it lies directly to the south of Bal Harbour. Haulover Park is separated from Bal Harbour by Bakers Haulover Inlet, and beach fill placements along the park appear to have little direct effect on the Bal Harbour shoreline.

Beach fill construction within these adjacent areas consists of the following events. Initial construction of the Surfside/North Miami Beach segment and the Haulover Beach Park segment of the Dade County BEC & HP was completed in 1978. One renourishment of Surfside was performed in 1999, and four renourishments of Haulover Park were performed, in 1980, 1984, 1987, and 1994. Offshore borrow areas were used as a source of fill for the initial construction of Surfside/North Miami Beach and Haulover Park, and for the first renourishment of Surfside. Material removed from the maintenance dredging of Bakers Haulover Inlet was used as the source of fill for all four renourishments of the Haulover Park shoreline. Each of these events are described in greater detail in table 14 below.

Table 14
Surfside, Haulover Park Beach Fills

Date	Volume Placed (cy)	Berm Width, ft	Comments
	Surfside		
1978	2,640,000	130	Init constr : Surfside, N Miami Bch
1999	590,000	250	1st renourishment
	Haulover Park		
1978	300,000	50	Initial construction : Haulover
1980	43,163		Maint dredging – Haulover Inlet
1984	35,000		Maint dredging – Haulover Inlet
1987	235,000		Maint dredging – Haulover Inlet
1994	24,560		Maint dredging – Haulover Inlet

Site Description.

The project area is located along the northernmost 0.85 miles of shoreline along the barrier island which extends from Government Cut (Miami Harbor) northward to Bakers Haulover Inlet. An aerial photograph of the Bal Harbour shoreline is provided in figure 15. This reach of shoreline is completely developed with high-rise condominiums and hotels, as shown in the photographs in figures 16a and 16b. The shoreline consists of an open sandy coast, with dense vegetation planted by the city along the back-beach area. This area has been developed by the city into a park which is widely used and contributes greatly to the area's aesthetics. Recent site inspections revealed that the vegetated area of the park has grown seaward to the point where it mostly covers the 95-foot width of the design berm along the length of Bal Harbour. This vegetated area contains a variety of tropical foliage including dense areas of coconut palms, sea oats, and sea grapes. Nature trails, benches, fences, a sprinkler system, and other park facilities are located within this area (figure 16c).

Beach widths along Bal Harbour vary greatly according to position along the shoreline, and also with time relative to the last beach renourishment project. Historically, erosion rates are higher along the northern half of Bal Harbour and as a result beach widths are the narrowest in this region. Following each beach renourishment project erosion is usually noted along this northern area first. Each full-scale beach renourishment results in the re-construction of a 240-foot wide berm along Bal Harbour (as measured from the ECL, which is located along the western edge of the vegetated zone). The recently-completed 2003 Bal Harbour renourishment is shown in figure 16d.

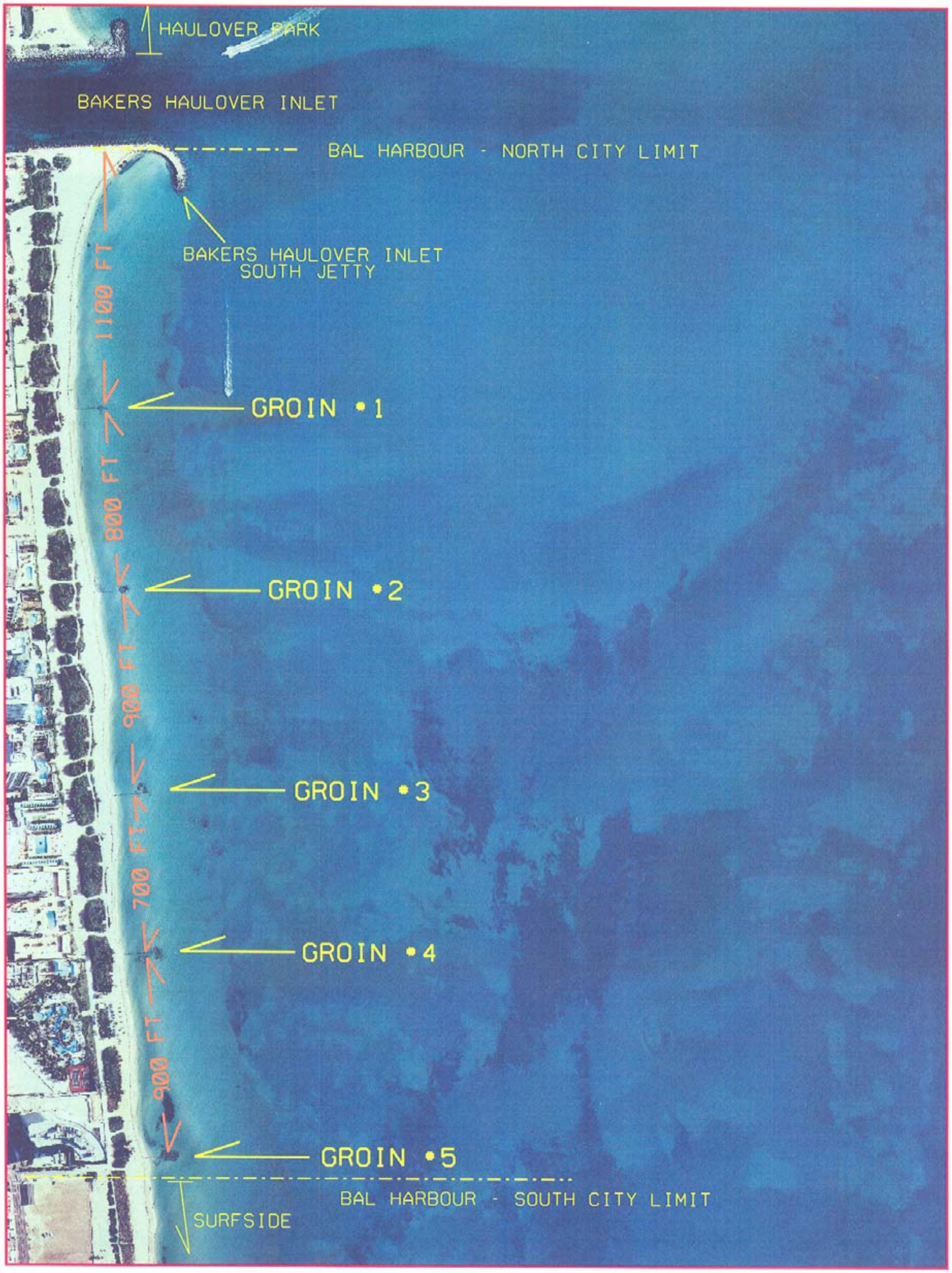


Figure 15. Bal Harbour Shoreline & Existing Structures.

Figure 16a. Upland development along Bal Harbour, August 2004.

Figure 16b. Upland development along Bal Harbour, August 2004.

Figure 16c. Vegetated park area along upland portion of berm, Bal Harbour.

Figure 16d. Recently-completed renourishment, September 2003, Bal Harbour.

Figure 16e. Eroded area at northern Bal Harbour, January 2004.

Figure 16f. Eroded area at northern Bal Harbour, August 2004

Figure 16g. King pile groins, buried by recent renourishment, January 2004.

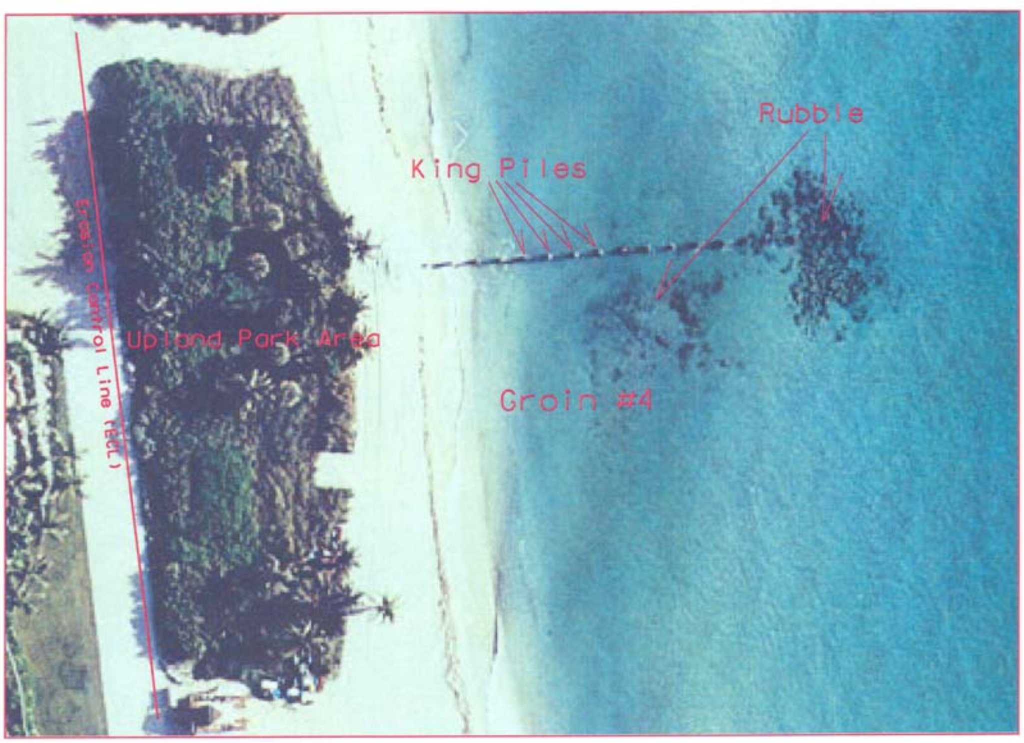


Figure 16h. Typical king pile groin, aerial view, with beach in eroded condition.

Prior to each periodic renourishment, berm widths can be as little as 100 feet from the ECL (immediately in front of the seaward edge of vegetation), occasionally resulting in minor erosion into the design template. The northern half of the Bal Harbour shoreline usually reaches this critical point of erosion long before the southern half. This process can be seen in its initial stages in figure 16e. This photograph was taken in January 2004, six months following completion of the Bal Harbour 2003 renourishment. At that time the berm had receded about 75 feet, along an area centered about 500 feet south of the north end of Bal Harbour. The wider berm along southern Bal Harbour can be seen in the distance. Figure 16f shows the same area in August 2004, when northerly transport of sediment during the summer months had filled the eroded area to a large extent.

Five king pile groins were constructed along the length of Bal Harbour prior to construction of the Federal project. No construction plans or design data for these structures is available, either through the agency representing the local sponsor (DERM : Dade Environmental Resource Management) or through Bal Harbour's engineering department. Anecdotal evidence suggests that these structures were built in the 1950's. Each of the groins is similar in design, and consist of concrete king piles (slotted piles) between which horizontal panels can be placed to form a barrier. The tops of these piles are shown in figure 16g. It is not known how many of the horizontal panels between the king piles may still be in place - at this time all five groins are completely buried from the 2003 Bal Harbour renourishment.

The king piles are driven in a line extending across the beach face at 10-foot centers. The top elevation of the seaward portion of the groins is about mean high water. The landward portions of these structures remain buried under the beach most of the time, so the exact dimensions and conditions of these portions of the structures can not be readily determined by site inspection. Piles of rubble have been placed around the seaward tips of the five structures, presumably as scour protection. Some photographic evidence suggests that the rubble may extend along the length of the groins as well (see figure 16h). Following each renourishment of the Bal Harbour shoreline, all five groins are buried completely by the 240-foot wide construction berm.

Throughout the remainder of this report these relic groins will be numbered 1 through 5 proceeding from north to south along the Bal Harbour shoreline, as shown in figure 15. The lengths of each groin as measured from the ECL, and from the seaward edge of the vegetation line are shown in table 15 below. The spacings between the existing king pile groins are as follows : 1,100 feet between the Bakers Haulover Inlet south jetty and groin 1; 800 feet between groins 1 and 2; 900 feet between groins 2 and 3; 700 feet between groins 3 and 4; and 900 feet between groins 4 and 5.

Table 15
Lengths of Existing King Pile Groins at Bal Harbour

Structure	Length (ECL)	Length (from veg line)
Groin 1	320	220
Groin 2	320	190
Groin 3	320	205
Groin 4	250	145
Groin 5	195	85

Discussion of Physical Process - Bal Harbour Study Area.

General. The physical data presented in the preceding regional analysis section of this report will be examined in greater detail to better understand the physical processes within the Bal Harbour study area. The causes of the observed accelerated erosion along the shoreline will be examined in detail. Once the physical processes along the study area are understood, reasonable alternative plans of improvement can be developed to reduce the erosional losses from this area.

Volumetric and Shoreline Position Changes. Analysis of the measured volumetric and shoreline position changes presented previously in this report can provide valuable information for determining shoreline trends in the historic era (pre-Federal project) vs the present era (post-Federal project), and can provide a baseline for measuring the effectiveness of any proposed plans of improvement.

As seen in table 8, prior to construction of the Bakers Haulover Inlet channel in 1925 the Bal Harbour shoreline generally accreted, whereas after the construction of the inlet the shoreline generally has eroded. This transition from an accretionary shoreline to an erosive shoreline is primarily due to the interruption of littoral transport caused by construction of the inlet. The historical (pre-project) rates in table 8 are based on the time interval from 1883 to 1919, and the average rate of shoreline change along Bal Harbour (profiles 18 and 19) was about +10,000 cy/yr (+2.2 cy/lf/yr) during that period. Prior to 1925 no inlet existed at this location, and sediment flowed uninterrupted along this reach of coast. Following the construction of the inlet by local interests in 1925, littoral transport was disrupted and downdrift erosion began to occur south of the inlet along the Bal Harbour shoreline. The right-hand side of table 8 provides average annual volumetric changes following inlet construction, over the interval from 1927-1961. As seen in the table, long-term erosion was noted along Bal Harbour during this time, at an average annual rate of –32,000 cy/yr (–7.1 cy/lf/yr).

The Federal BEC & HP project was authorized and constructed in response to large-scale erosion along the Dade County shoreline, including Bal Harbour. The left side of table 9 shows that during the period following construction and initial fill stabilization, volumetric change rates along Bal Harbour were reduced to about –27,850 cy/yr (–5.4 cy/lf/yr) for the period 1990-2002, mainly though the placement of large-scale beach fills. It is important to note that beach renourishment projects were constructed along this area in 1990 and 1998. As a result, the shoreline was in a fully renourished condition at the start of the period of analysis and was in a nearly fully-eroded condition at the end of the period of analysis, which accounts for the observed erosion with the Federal project in place. Had the survey interval extended between similar points in the renourishment/erosion cycle (ie post-construction to post-construction) the measured erosion rate would be near zero. With fill placements factored out entirely (right side of table 9) the measured erosion rate along Bal Harbour increases to –55,620 cy/yr (–10.8 cy/lf/yr). This value will be defined as the existing (post-Federal project) background erosion rate.

From the preceding discussion it is noted that the pre-project erosion rate along Bal Harbour was –32,000 cy/yr. This value is compared to the post-project erosion rate of near zero with the ongoing placement of beach fills, or –55,620 cy/yr with beach fills factored out. It is apparent that the present background erosion rate along Bal Harbour is higher than the pre-project rate. This may be due to any of the following factors. First, erosion rates will fluctuate somewhat from one renourishment cycle to the next in response to the different wave (and other environmental) conditions which occur over that particular time interval. This survey data spans only one complete renourishment/erosion cycle; data from multiple cycles will be required to establish an accurate average background erosion rate. Secondly, higher than normal losses are always experienced during the first few years as newly-placed beach fills reach their new cross-shore equilibrium. Finally, accelerated losses may occur following beach fill construction due to end losses from the greatly increased berm width. In the case of Bal Harbour, relatively large volumes of fill could be lost around the seaward tip of the short jetty at the north end of the project, as well as at the south end of the fill.

Survey data indicates that the Bal Harbour shoreline does not erode in a uniform manner. The non-uniform distribution of erosion along the Bal Harbour shoreline can be seen in the unit erosion rates on the right side of table 9. Erosion rates at the north end of the study area (R-27) are relatively low, increasing rapidly to a maximum about 1,000 feet south of the inlet (between R-28 and R-29), then gradually decreasing with distance from the inlet. These measured erosion rates from table 9 are plotted graphically in figure 17 below. These plotted values represent normalized erosion rates in units of cubic yards per foot per year, and the effects of beach fills have been removed from the data presented in this graph.

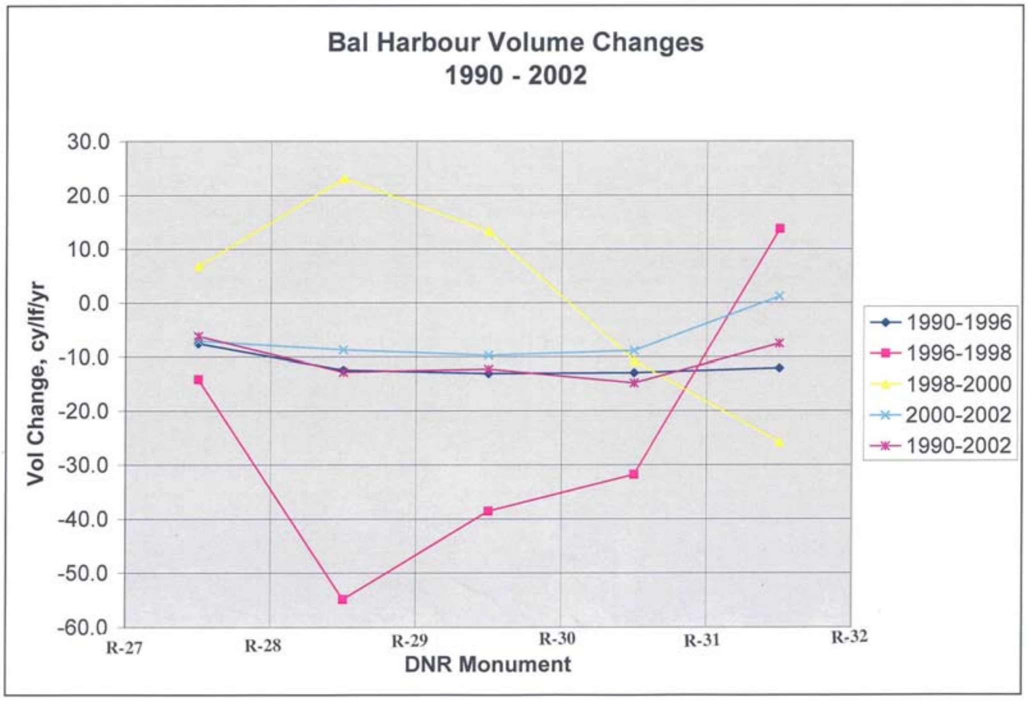


Figure 17. Plot of Unit Volumetric Changes – Bal Harbour (1990 – 2002).

These plotted values demonstrate several of the trends described above regarding volumetric changes along the Bal Harbour shoreline. First, it is seen that a high degree of variability exists in erosion rates along the Bal Harbour shoreline. Data from two time intervals (1990-96 and 2000-02) fall very close to the long-term average values, but the intervals 1996-98 and 1998-2000 show a much wider distribution of unit erosion values. Data acquired during the survey interval 1998-2000 even shows a reverse trend : erosion along the southern reach of Bal Harbour; accretion along the northern reach (possibly a result of sediment transport reversal during the summer months). Despite these variances, observations dating back to the construction of the inlet in 1927 confirm the general trend seen in the survey data : higher erosion rates occur immediately downdrift of the inlet, with erosion decreasing with distance to the south. This trend is usually an indication of the interruption of longshore transport caused by the adjacent inlet.

Littoral Transport Rates. As discussed in previous sections of this report, sediment transport along the coastline is highly seasonal and can also vary greatly from year to year. The sediment budget developed by CSI, Inc in 1997 (reference 2d) provided a detailed description of the littoral processes along the Dade County and southern Broward County shorelines. This analysis is still considered to accurately represent littoral transport rates along these reaches of shoreline. According to the CSI analysis the long-term net littoral transport in the project area is southward-directed, at an average rate that varies from about 60,000 cy/yr immediately north of Bakers Haulover Inlet to 19,000 cy/yr at the inlet, increasing to 48,000 cy/yr at Bal Harbour's southern city limit.

These rates imply that two processes are occurring : first, southward-directed sediment north of the inlet is being diverted at the inlet, at a rate of about 41,000 cy/yr. This material is being deposited into the inlet's ebb and flood shoals at rates of about 32,000 cy/yr and 9,000 cy/yr, respectively. These rates are verified by long-term dredging records. The second conclusion to be drawn from an analysis of the CSI sediment budget is that material is being eroded from the Bal Harbour shoreline at a rate of (48,000-19,000 cy/yr) = 29,000 cy/yr.

A discrepancy exists between the erosion rate computed in the CSI sediment budget along Bal Harbour (-29,000cy/yr) and the updated average erosion rate computed in this report (-55,600 cy/yr). Both of these values were derived directly from analysis of survey data, and can only be attributed to the variability of erosion rates with time, which is demonstrated in figure 17. The CSI sediment budget was derived from an analysis of beach profile survey data extending from 1980 to 1996; the sediment budget presented in this report updates the CSI analysis with survey data extending from 1990 – 2002, including two high resolution county-wide lidar surveys.

The 1997 CSI analysis was developed based on the assumption that all volume changes along the shoreline are due to gradients in the longshore transport rate. This assumption is not entirely valid along the Bal Harbour shoreline. The wave refraction analysis contained in this study shows that a nodal point exists along the northern third of the study area. Sediment transport north of this point is nearly always directed northward; sediment transport south of this point is directed southward (for northerly wave events). The material transported northward of the nodal point is not accounted for in the CSI sediment budget, but will be included in the updated budget as a result of numerical modeling in this report.

An analysis of survey data provides an approximation of the net movement of sediment in the study area, but survey data alone can not provide information on the detailed movement of sediment around the inlet and ebb shoal system. An examination of the aerial photograph in figure 2 and the lidar bathymetry shows that the ebb shoal extends around Bakers Haulover Inlet, providing a continuous path for sediment transport. Transport of sediment across this shoal onto the Bal Harbour shoreline is impeded to some degree by the relatively deep waters of the channel, and by the depths along the southern lobe of the shoal. According to the CSI sediment budget, 19,000 cy/yr bypasses the inlet and is deposited onshore along the Bal Harbour shoreline – a value based entirely on volumetric changes along the region's shorelines and the inlet shoals. Numerical modeling will be used in this report to expand on this analysis of sediment movement along the Bal Harbour shoreline.

The littoral transport rates developed in the CSI report are widely accepted as representative of large-scale sediment transport along the Dade/Broward coastlines, and much of the data from that study will be used in this report to update and expand the analysis of physical processes in the region of Bakers Haulover Inlet. Previous sediment budgets have made no allowance for the sediment transport reversal along northern Bal Harbour, and the numerical modeling presented later in this report will attempt to define the littoral processes in the vicinity of the inlet in greater detail. To achieve this, the numerical model STWAVE will be used to conduct a wave refraction analysis to quantify the effects of the offshore bathymetry on wave refraction patterns throughout the study area. Output from STWAVE will be entered into the numerical shoreline change model GENESIS to calculate shoreline change patterns and littoral transport rates along the length of the Bal Harbour shoreline.

In order to properly calibrate the GENESIS model several existing conditions must be defined, among these are the littoral transport rates and shoreline changes at the model boundaries and volumetric changes within the study area. The transport rates at each end of Bal Harbour as developed in the CSI study are considered to be accurate and will be used in the numerical model calibration. Specifically, the calculated net transport rate of 48,000 cy/yr (southward) at the southern city limit of Bal Harbour will be used, and the bypass rate of 19,000 cy/yr around Bakers Haulover Inlet and into Bal Harbour will also be used. The updated net annual volume change as developed in this report is –55,620 cy/yr along Bal Harbour. The distribution of erosion as shown in figure 17 is also an important factor which will be used in GENESIS calibration.

A partial sediment budget has been developed in the preceding discussion of littoral processes. In order to better define the complex processes in the vicinity of the inlet numerical modeling is required, and this modeling will be based on many of the parameters developed in this study up to this point. A more detailed discussion of the sediment budget within the study area will be provided in later sections of this report, based on the results of the following STWAVE and GENESIS numerical modeling studies.

NUMERICAL MODELING

Wave Refraction Analysis.

Overview. As seen in the previous section of this report, the wave environment which drives the littoral processes along the Dade County shoreline can be characterized as relatively low energy compared to most of the east coast of the United States. However, the region offshore of Dade County contains several large-scale features which tend to refract waves in an irregular manner, possibly creating areas of wave energy focusing along the shoreline. This wave energy focusing can in turn result in areas of localized erosion. It has long been suspected that this phenomenon is at least partially responsible for several areas of persistent localized erosion which have been observed since the construction of the Dade County BEC & HP project. The 2001 Dade County Evaluation Report (reference 1g) was the first study to use lidar bathymetry of the region to perform a county-wide wave refraction analysis. That study identified a region of wave energy focusing along the Bal Harbour shoreline created primarily by refraction around the Bakers Haulover Inlet ebb shoal. This study will use expanded and updated lidar coverage to conduct a more site-specific refraction analysis of the region offshore of Bal Harbour.

Snell's Law states that waves propagating from deep water into shallower waters will tend to align parallel to the offshore depth contours. Two large-scale offshore features exist near the Bal Harbour project area which cause the bathymetry to significantly depart from the straight and parallel depth contours observed along most of Florida's coastlines : the offshore coral reef system, and the ebb shoal at Bakers Haulover Inlet. These areas are shown graphically by a plot of the November 2002 lidar bathymetry along the central Dade County coast in figure 18. The shore-parallel reefs and ebb shoal at Bakers Haulover Inlet are obvious in this figure. In addition, strong tidal currents through Bakers Haulover Inlet also have an effect on the refraction of incoming waves. These features can significantly alter the directions and amplitudes of incoming deepwater waves and may result in significant wave energy focusing along the beach. In this analysis the term "wave energy focusing" will be used to denote all increases in wave height due to changes in wave direction (refraction), and/or changes in amplitude as waves enter shallower waters (shoaling). The term "wave energy dissipation" will denote areas where wave heights have been reduced through refraction and/or shoaling processes.

Dade County's offshore region is dominated by an extensive system of coral reefs. These reefs extend in a roughly shore-parallel direction along the entire southeast coast of Florida from north of Palm Beach County through Broward and Dade Counties, continuing southward into Monroe County (Florida Keys). As seen in figure 18, three primary lines of reef extend along the length of Dade County's shoreline. These reefs rise from less than one foot to over 10 feet above the surrounding areas. Depths along the reefs are irregular in the longshore direction as well, with alternating high and low areas and occasional breaks in the reef line. In addition, the reefs do not extend in a straight line : although the large-scale orientation is shore-parallel, frequent undulations in the reef line occur. Waves passing over

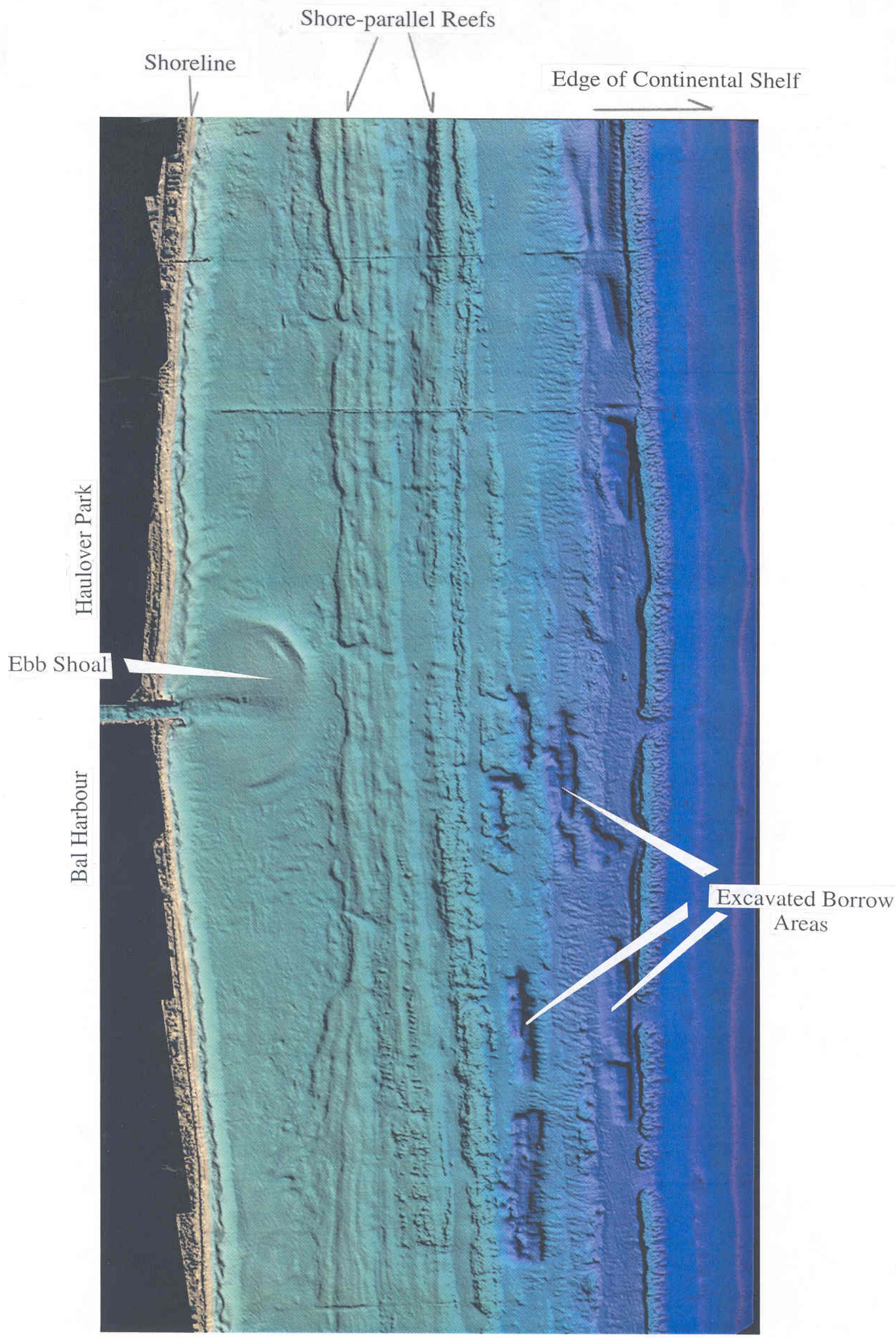


Figure 18. Display of November 2002 LADS bathymetry.

the reef system are refracted in a complex manner in accordance with the configuration of the reef's bathymetry, and as a result wave energy may be focused along some areas of the shoreline, and spread out or diffused over other areas.

Aside from the coral reef system, the ebb shoal at Bakers Haulover Inlet is the second most dominant feature offshore of the study area. The shoal is a large dome-shaped feature roughly one mile in diameter, located primarily on the north side of the Bakers Haulover Inlet entrance channel. The crest of the shoal is less than 10 feet deep at a distance offshore where average depths along the Dade County shoreline range from about 20 to 25 feet. Waves passing over this feature tend to be refracted toward the inlet. In addition, tidal currents through Bakers Haulover Inlet have an effect on wave refraction. The strong ebb tidal currents through the mouth of the inlet also tend to refract waves toward the inlet, while flood tides tend to refract waves away from the inlet. These processes will be discussed in detail in the following section and in Appendix C.

With the advent of lidar technology the complicated reef structure could be surveyed in greater detail than ever before. Soundings from the June 2000 SHOALS survey and the November 2002 LADS survey were each spaced an average of about 4 meters (12 feet) apart over the entire survey area. The level of detail provided by lidar technology is shown in the graphical plot of a portion of the 2002 lidar survey in figure 18. Large-scale features such as the shoreline, reef-lines, the Bakers Haulover Inlet and ebb shoal complex, and the edge of the continental shelf are easily seen. The shore-parallel trenches observed between the second and third reefs are the excavated borrow areas used in the original construction of the project. Some small-scale features such as individual palm trees on the beach and even offshore shipwrecks may also be seen in figure 18, but are better seen by zooming in on the digital data set. Both lidar surveys extended along the entire length of the Dade County shoreline. Soundings extended approximately 4,000 feet seaward of the shoreline for the June 2000 SHOALS survey, which was used as a basis for the refraction analysis presented in the Jacksonville District's 2001 Evaluation Report. The bathymetry from the November 2002 LADS lidar survey extended over 2.5 miles seaward from the shoreline; this data was used in the updated and expanded wave refraction analysis contained in this report.

The November 2002 LADS survey extended from the shoreline to beyond the edge of the continental shelf into deep water, providing ideal bathymetric data for a wave refraction analysis. Since wave transformations are usually more pronounced in shallower water (particularly for the short-period waves which regularly occur off Dade County), the detailed bathymetry provided by these lidar surveys allowed a more comprehensive wave refraction analysis to be performed than ever before. The numerical modeling presented in this section summarizes the methodology used to quantify the effects of the offshore bathymetry on wave energy focusing along the Bal Harbour shoreline.

Description of Numerical Wave Transformation Model. The numerical wave transformation model STWAVE was used in this study for modeling changes to input wave fields as they pass over the irregular offshore bathymetry of Dade County. STWAVE (STeady-state spectral WAVE) is a gridded steady state spectral wave transformation model for the near-coastal zone. STWAVE accepts arbitrary directional wave spectra specified at an offshore boundary and propagates the spectral energy in the onshore direction, across bathymetry specified by the user. Distribution and dissipation of wave energy is based on principals of linear theory that include source/sink terms for wind, wave-wave interactions, wave-current interactions, wave-bottom interactions, and wave breaking based on water depth and wave steepness. Due to the nature of the solution scheme, spectral energy is limited to propagation in a half-plane directed toward shore. All energy propagated offshore is neglected.

STWAVE has several inherent limitations. For the best application of the model to wave transformation problems, the following assumptions must be made:

a) Mild bottom slopes.
b) Negligible wave reflection.
c) Spatially homogeneous offshore waves.
d) Steady state waves, winds, and currents.
e) Linear refraction and shoaling with negligible bottom friction effects.
f) Linear wave-current interaction.

Criteria (b), (c), (d), and (f) are met within this the study area. Criteria (a) and (e) are satisfied the majority of the time, and/or along the majority of the study area. The 'mild bottom slopes' specified in criteria (a) exist throughout the entire study area; the only exceptions are along some portions of the edges of the reefs where limited reaches of steep, vertical, or undercut dropoffs exist. These near-vertical areas are small in extent throughout the project area and in most cases will be too deep to be 'felt' by the short-period waves which dominate the local wave environment. The overall impacts to the wave transformation process will be minor, but for long period waves passing over such vertical areas the predicted wave amplitudes may be slightly greater than the actual amplitudes.

Criteria (e) may have a stronger impact on predicted vs actual nearshore wave energy. Most of the surface area of the coral reefs offshore of Dade County are covered with marine growth, consisting of 'soft' organisms such as sponges, sea fans, etc, and 'hard' organisms, primarily numerous types of hard corals. These organisms typically grow to about 1 to 2 feet high along the hardbottom areas, and will increase bottom friction, particularly for longer-period waves passing over these vegetated reef areas. In addition, many of the small-scale features of the reef which contribute to roughness are smoothed out by the 4-meter spacing of the lidar survey elevation points, and during the depth-averaging routine which is performed during formation of the uniform (25-foot) grid for the STWAVE analysis.

However, the effect on modeled vs actual propagated wave energy of are thought to be relatively small since the most heavily vegetated areas are in deeper waters where bottom friction exerts less influence on wave energy. In addition, any reduction of wave heights caused by bottom friction should be uniformly distributed along the shoreline, so the relative effects of wave energy focusing along the shoreline (which are critical to the prediction of erosive areas) can still be seen.

STWAVE is a finite-difference model that employs a backward ray-tracing scheme to calculate wave spectra across a rectangular grid. The following inputs are required to run the STWAVE model:

a) Bathymetric data
b) Size and resolution of the computational grid
c) Directional wave spectra at the offshore boundary
d) Wind speed and wind direction
e) Current field (if applicable)
f) Water level adjustment (if applicable)

Model output includes wave height, peak spectral period, and mean wave direction given at all grid points. The same parameters as well as a directional spectrum can also be generated for selected grid points. More detailed information regarding STWAVE can be found in reference 3l (Smith/Resio/Zundel).

STWAVE Input Data. Bathymetry for the wave refraction analysis was obtained entirely from the November 2002 LADS lidar survey. This survey provided the most detailed bathymetric data ever gathered across the width of the Dade and Broward County continental shelf. This lidar data set extends from landward of the dune line to a distance offshore of about 2.5 miles, which corresponds to water depths of about 200 feet. Elevation data consists of random scatter points spaced at roughly 4-meter (12-foot) intervals throughout the survey area, which extends from northern Broward County to southern Dade County. This bathymetric data was sufficiently detailed to capture both the relatively straight and parallel contours found in the deeper areas and the complex contours of the shallow hard bottom reef systems characteristic of the south Florida nearshore zone.

Numerical wave transformations in STWAVE are based on bathymetric data projected onto a regular Cartesian grid (see figure 19). For the best results the STWAVE grid is oriented such that the y-axis is aligned parallel to the offshore depth contours and the x-axis is aligned normal to those contours. Based on the bathymetry of the Dade County coastal zone the optimum orientation of the grid was achieved by aligning the y-axis directly north to south with the grid x-axis roughly normal to the coastline. The STWAVE computational grid was generated using the SMS (Surface-water Modeling System) software package developed by Brigham Young University and the U.S. Army Corps of Engineers' Coastal Hydraulics Laboratory (reference 3k). The grid was dimensioned with 630 cells in the cross-

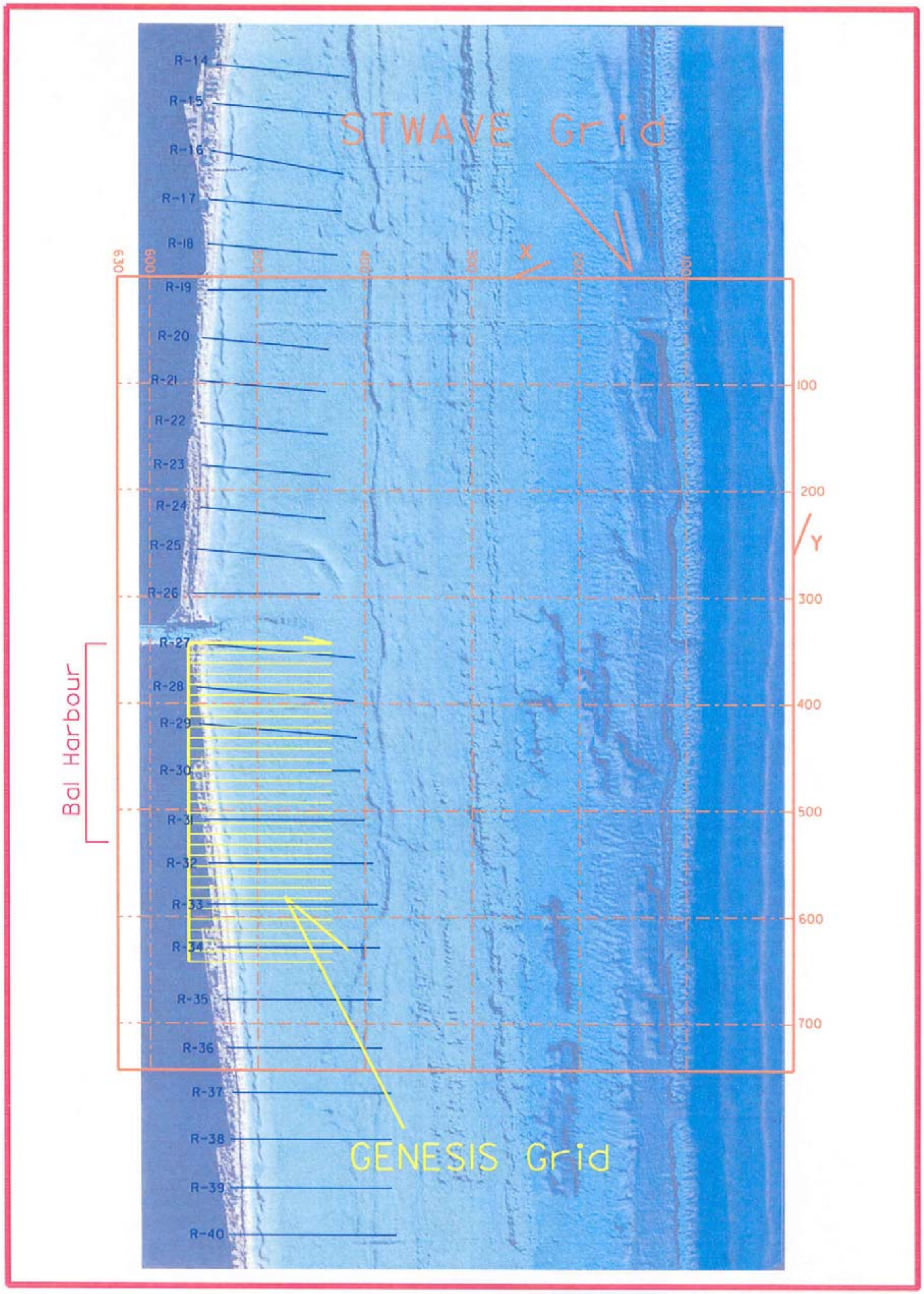


Figure 19. STWAVE and GENESIS computational limits, bathymetry.

shore direction and 744 cells in the longshore direction, with a grid cell resolution of 25 feet in both the x- and y- directions. This produced an area of coverage measuring 3.0 miles from west to east, extending from the upland area seaward to approximately the 200-foot depth contour, and 3.5 miles north to south, with Bal Harbour located near the center of the grid. The total number of computational cells in the grid is therefore 630 x 744 = 468,720. Figure 19 shows the computational limits and bathymetry within the STWAVE grid, as well as the layout of the GENESIS shoreline modeling grid (to be discussed later). The locations of DNR monuments are provided for reference. Note the location of Bal Harbour in reference to both model grids.

Wave and wind conditions used to develop STWAVE model input parameters were obtained from the WIS station 9 database described previously in this report. WIS station 9 is located at latitude 26.0 degrees and longitude 80.0 degrees, in 220 meters (722 feet) of water, so the wave record can be considered to be representative of deepwater wave conditions. A total of 58,440 wave and wind conditions are included in the complete WIS station 9 record, which consists of a time series of wind/wave events at 3-hour intervals between 1976 and 1995. Given the extent and resolution of the STWAVE grid, such a large number of conditions is prohibitive as model input.

Relatively wide variations in wave energy distribution occur from year to year within the WIS wave record. For purposes of this modeling effort it was determined that conditions with the highest probability of occurrence would provide the most insight into areas of wave energy focusing along the project area. Wave events with the highest frequency of occurrence would presumably be responsible for a large portion of sediment transport along the coast. In addition, strong (but relatively infrequent) storm events are also known to result in large volumes of sediment transport, and these events will be examined separately. Two sets of input wave conditions were therefore developed : one based on “average” conditions and one based on “storm” conditions.

<u>“Average” Input Wave Conditions</u>. The “average” wave conditions were derived in the following manner. Due to the known seasonal fluctuations of wave parameters along the Dade County coast, it was determined that average wave conditions would be formulated for each month of the year. This was accomplished by grouping all 20 years of WIS data into 12 monthly files : all “January” wave data was consolidated into one file, “February” data into a separate file, etc. Each file was then analyzed statistically to determine the most representative conditions for each of the 12 months, using the following methodology:

In order to summarize and consolidate the information into a manageable number of representative model runs the 20 years of hindcast data (sorted by month) were separated into 10-degree direction bands and further separated into 3.3-foot (1meter) wave height bands. This resulted in a total of 105 direction-height bands. The number of wave conditions per band as well as the average wave height, average peak period, average wind speed, and average wind direction were calculated for each of the direction-height bands.

Deepwater wave parameters (wave height, peak period, wind speed, and wind direction) were then extracted from each direction-height band within the STWAVE half-plane. Wave direction for each case was taken to be the median value of the wave direction band it fell under (i.e. band 0-10 degree resulted in a wave direction of 5 degrees). A summary of the resulting deepwater wave and wind conditions is provided in Table 16.

Table 16
Averaged Monthly Wave Conditions from 1976 to 1995

Month	Wave Height (Ft)	Peak Period (Sec)	Wave Direction (Deg*)	Wind Speed (Ft/sec)	Wind Direction (Deg)
January	4.3	10	35	16.4	87
February	3.6	10	25	16.4	124
March	4.3	8	65	19.7	90
April	3.0	10	35	13.1	121
May	2.6	6	85	16.4	104
June	2.0	5	125	13.1	135
July	2.0	5	115	13.1	120
August	2.3	6	85	16.4	96
September	2.3	10	45	13.1	108
October	3.6	9	45	16.4	95
November	3.9	9	45	19.7	84
December	3.9	9	35	19.7	95

* Wave and wind directions are given in degrees clockwise from true north

Once representative wave conditions were obtained it was necessary to transform them from the WIS station 9 location in deep water (722 feet) to the shallower depth of the seaward boundary of the STWAVE computational grid (200 feet). Assuming straight and parallel contours between the depths, a Snell's Law routine was employed to transform waves from the WIS station location to the seaward edge of the STWAVE grid. Additionally, it was necessary to convert each set of wave parameters into a directional spectrum for model input. The input spectra were generated using a TMA spectral form with a spectral peakedness parameter, γ, ranging from 3.3 to 8.0 depending on the peak frequency, as described by Sorensen (reference 3i). A $\cos^n$ directional distribution was then applied to address directional spreading. The exponent n in this case ranged from 4 to 26 based on the peak wave period. The final spectra were comprised of 35 direction bands covering a half-plane in 5-degree increments and 50 frequencies ranging from 0.005 Hz to 0.495 Hz with a frequency increment of 0.01 Hz. Wave energy falling beyond +/- 85deg from shore normal was truncated. Since the monthly variations of wave energy distribution along the Bal Harbour shoreline are of interest, each of the 12 months of averaged data were separated into individual wave input files for STWAVE analysis. These monthly wave conditions were then numerically shoaled across the offshore bathymetry using STWAVE, and the results are presented in figures 20(a) through 20(l).

<u>"Storm" Wave Input</u>. In addition to simulating "average" conditions as depicted in figure 20, a separate analysis was conducted using typical "storm" conditions. The project area is subjected to large storm swells generated by 'northeasters' during the winter months. Many of these events are contained in the "average" wave-energy input files described above for the winter months. The storm events which were analyzed in this STWAVE study were selected from the WIS database due to their characteristics as "typical but severe" northeaster storms. Most northeasters generate large storm waves which can impact the coast for a period of a week or more, and occasionally severe long-duration northeasters cause erosional damages as great (or greater) than storm damages from hurricanes.

Due to the proximity of the Bahama Bank 60 miles offshore, only waves from extreme northerly directions can penetrate the Straits of Florida southward to the Dade County coastline. The erosive impacts of such storm waves on the Dade County shoreline are increased by the highly oblique angle at which these waves approach the shoreline. The storm parameters which were simulated in this STWAVE analysis were derived from two northeaster storms which occurred in January 1993. Storm #1 occurred on 3 January 1993, and storm #2 occurred on 21 January 1993. A summary of input wave parameters from both storms is provided in table 17 below.

Table 17
Input Storm Parameters for STWAVE Analysis

Event	Wave Ht (ft)	Period (sec)	Angle (deg)	Wind (mph)
Storm #1	6.9	7	82	29
Storm #2	10.8	9	69	22

These storm wave parameters were run on the same bathymetric grid as was used for the average monthly conditions above. STWAVE output for these simulations of storm conditions is provided at the end of Appendix C.

Discussion of STWAVE Results. Generally, as offshore deepwater waves pass into shallow water energy is lost through wave breaking, whitecapping, and bottom interactions. Wave direction is modified in a complex manner, with a large-scale trend toward aligning wave direction towards shore-normal as the wave fronts pass into progressively shallower waters. The Bakers Haulover Inlet ebb shoal appears to be the single feature with the greatest effect on wave refraction patterns in the Bal Harbour study area, and waves passing near this shoal are refracted in a complex manner which greatly increases wave heights along the shoal, and in most cases along the shorelines adjacent to the shoal.

As seen in the printout of each of the average monthly conditions in figure 20, the largest degree of wave energy focusing occurs along the northeast quadrant of the ebb shoal, corresponding to the area of shallowest depths along the shoal. This area of increased wave heights is located well seaward of the shoreline and has little direct effect on shoreline erosion, but two less-obvious areas of wave energy focusing occur directly along the shoreline immediately north and south of the inlet as a result of wave refraction around the shoal. The area of focusing which occurs along the northern third of Bal Harbour is of particular interest and will be explored further.

Figures 20(a) through 20(l) display model results for each of the 12 average monthly incident wave conditions listed in table 16. These figures display wave focusing in the form of wave height contours over the full computational grid, which extends from the north end of Haulover Park (near R-19) southward to the south end of Surfside (near R-36). Evidence of some degree of wave focusing along the northern Bal Harbour shoreline is clearly apparent in all cases. Wave focusing is marked by wave heights that are notably higher than those of surrounding areas, as shown by the color-coded scale which accompanies each refraction diagram.

The area of maximum wave energy focusing along the project area as seen in figure 20(a) – (l) occurs along the northern 2,000 feet of the Bal Harbour shoreline. This area of wave energy focusing consistently occurs along the same reach of shoreline and appears to be nearly independent of the incident wave condition. This region corresponds to the area between the northern two relic groins, extending northward to the Bakers Haulover Inlet south jetty. A similar region of wave energy focusing occurs north of the inlet. Examination of figure 2 shows that these regions of wave energy focusing along the shoreline north and south of the inlet correspond closely to the boundaries of the ebb shoal. Observations of erosion trends presented earlier in this report confirm that this area of the project experiences accelerated erosion relative to the adjacent beaches, and results of this wave refraction analysis indicate that wave energy focusing appears to be at least partially responsible for this erosion. The shoreline erosion in this region is obvious in figure 2.

An examination of the STWAVE output presented in figures 20(a) through 20(l) shows that as the wave fronts progress toward the shoreline for each monthly average wave condition, wave heights are reduced due to the various dissipative effects previously discussed. However, the wave heights over the ebb shoal are generally greater than or equal to the incident deepwater wave heights at the seaward edge of the model boundary. The refracted wave heights observed directly along the shoreline in the vicinity of the ebb shoal are somewhat less than the wave heights observed at the shoal but are still typically about equal to, or slightly less than, the incident deepwater wave height. Proceeding outward from the inlet (beyond approximately 2,000 feet) along the shoreline, breaking wave heights are considerably lower than in the vicinity of the inlet, and are much lower than the incident deepwater wave heights.

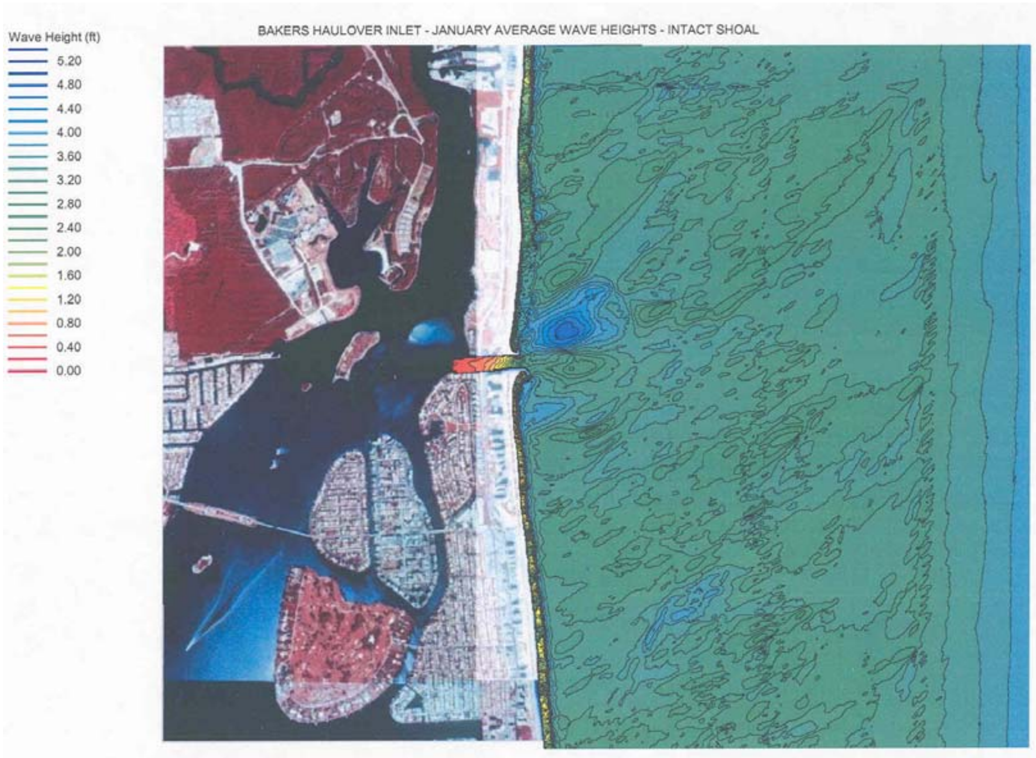


Figure 20(a). Wave Refraction Diagram – Average January Conditions
(Hs = 4.3 ft; T=10 sec; dir = 35 degrees)

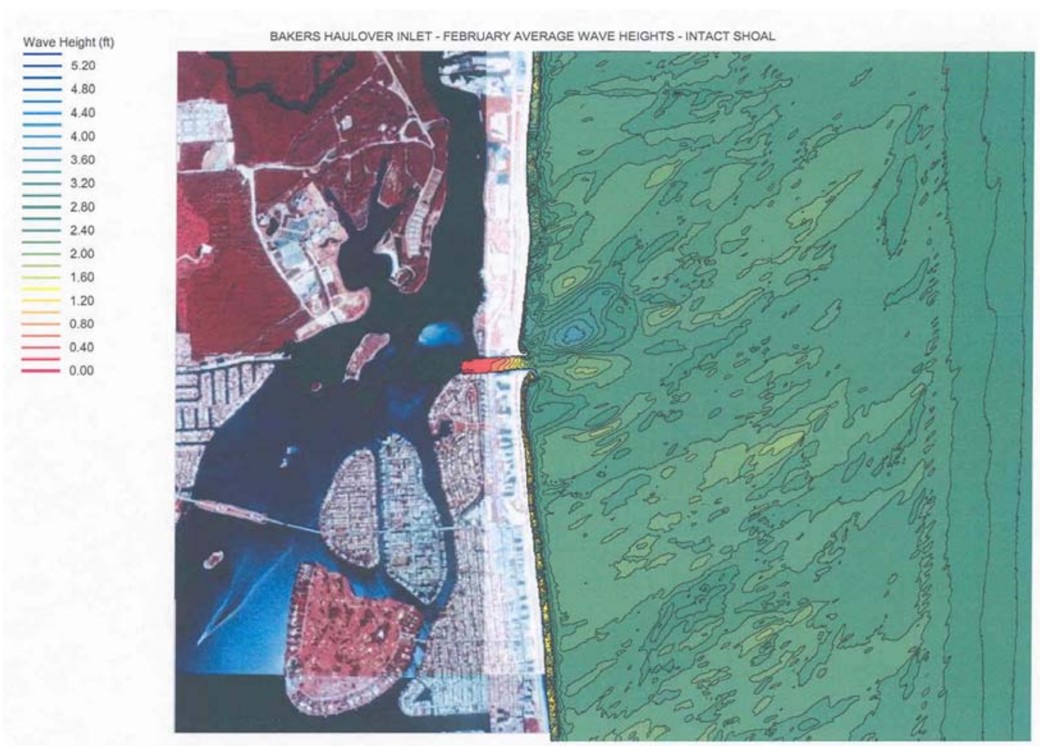


Figure 20(b). Wave Refraction Diagram – Average February Conditions
(Hs = 3.6 ft; T=10 sec; dir = 25 degrees)

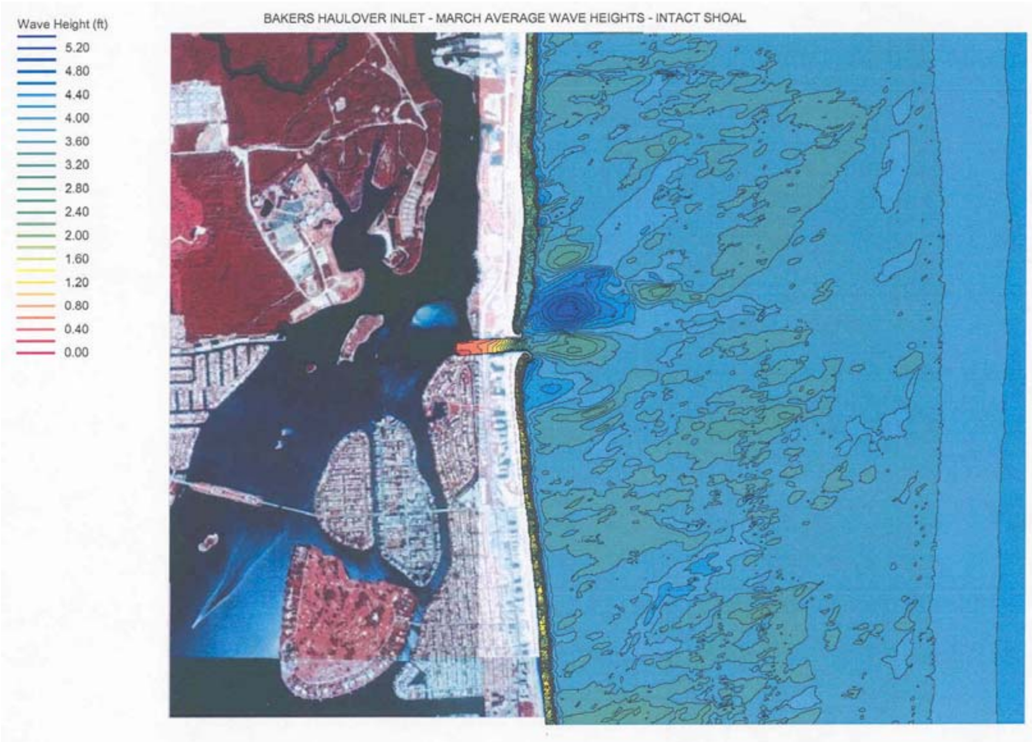


Figure 20(c). Wave Refraction Diagram – Average March Conditions
(Hs = 4.3 ft; T=8 sec; dir = 65 degrees)

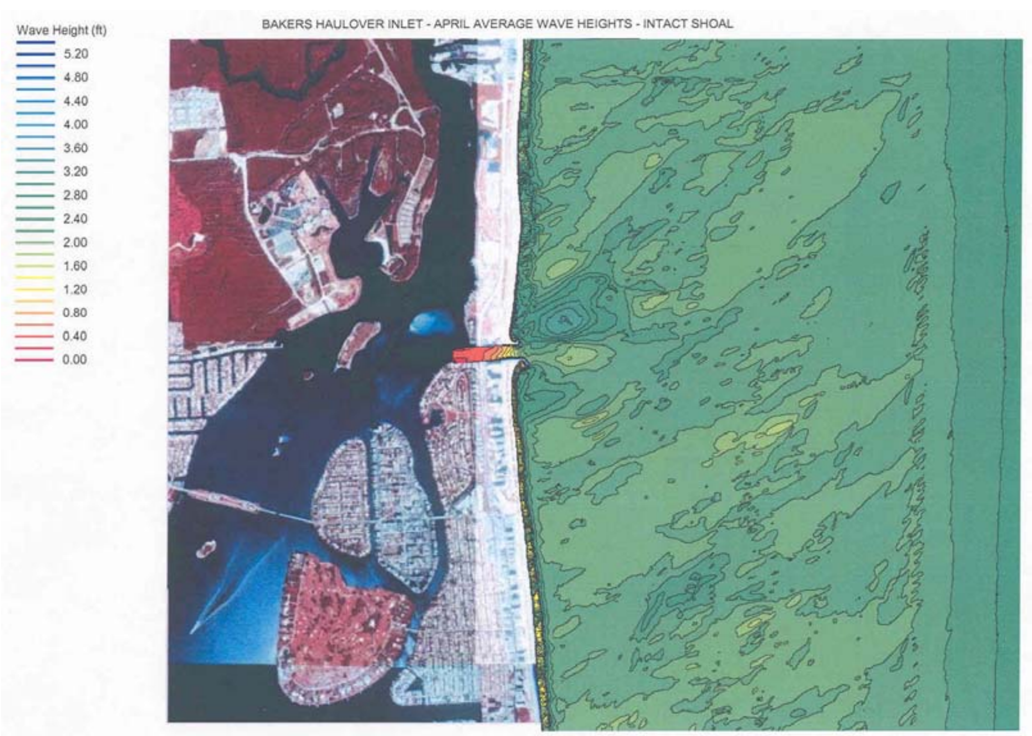


Figure 20(d). Wave Refraction Diagram – Average April Conditions
(Hs = 3.0 ft; T=10 sec; dir = 35 degrees)

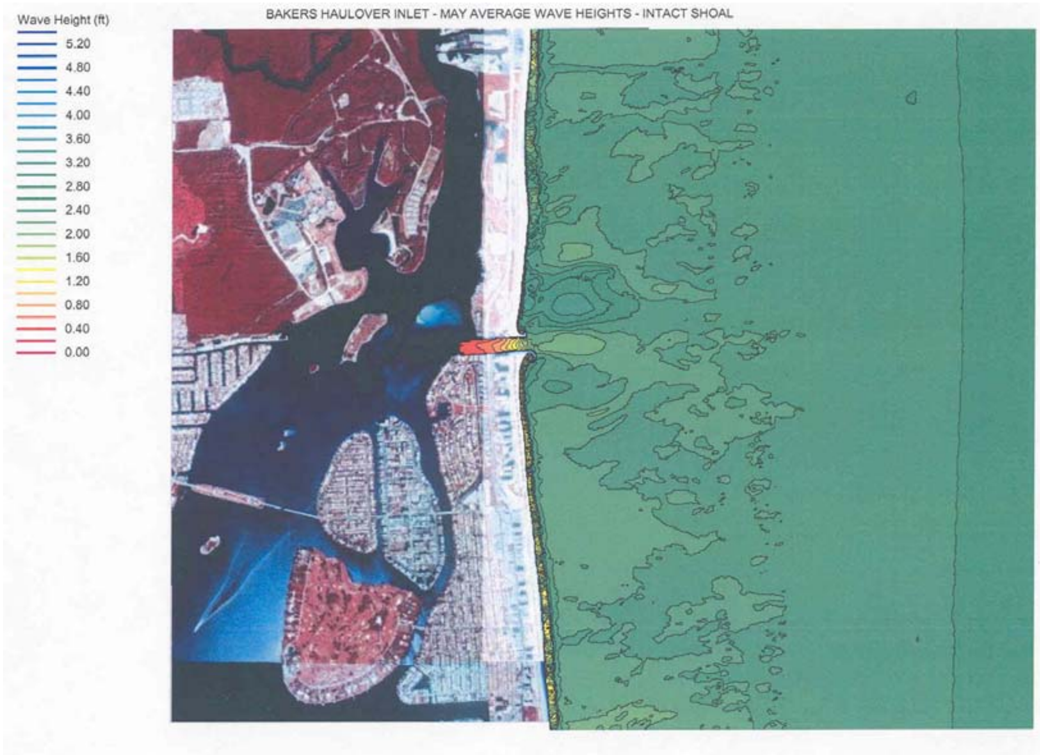


Figure 20(e). Wave Refraction Diagram – Average May Conditions
(Hs = 2.6 ft; T=6 sec; dir = 85 degrees)

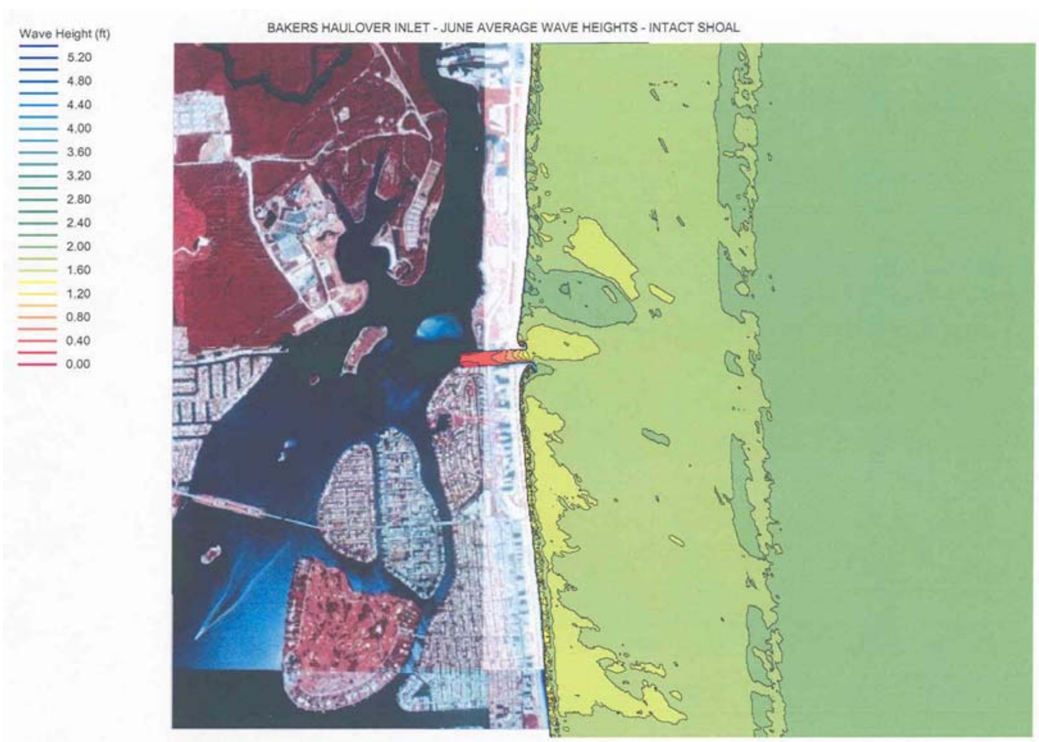


Figure 20(f). Wave Refraction Diagram – Average June Conditions
(Hs = 2.0 ft; T=5 sec; dir = 125 degrees)

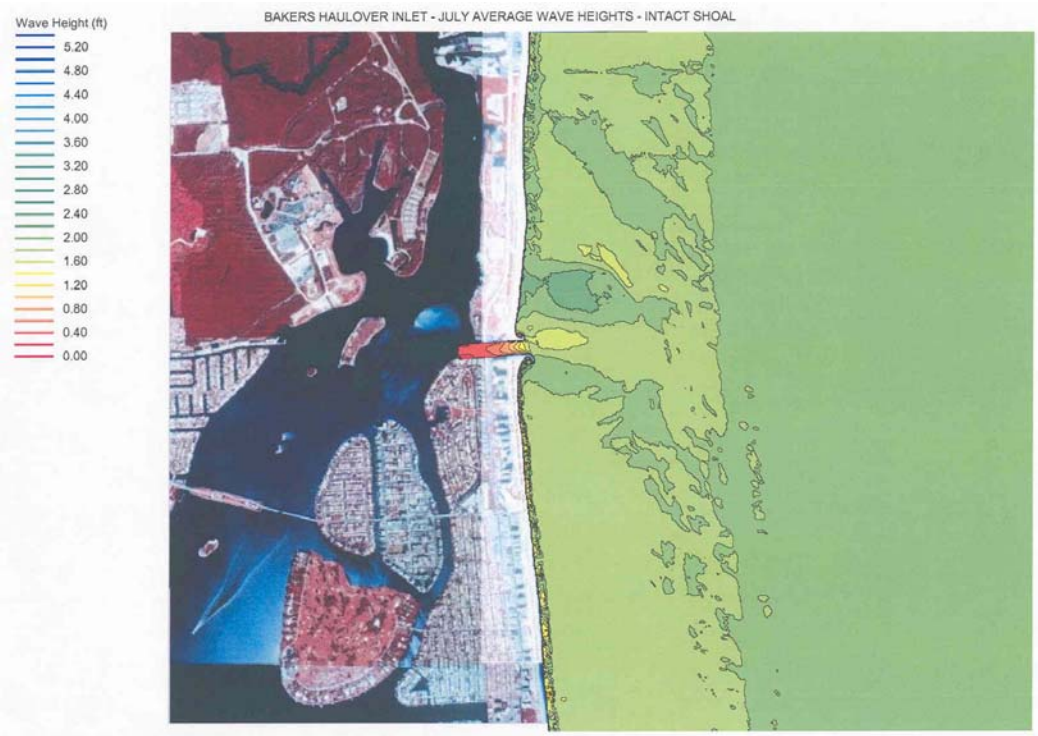


Figure 20(g). Wave Refraction Diagram – Average July Conditions
(Hs = 2.0 ft; T=5 sec; dir = 115 degrees)

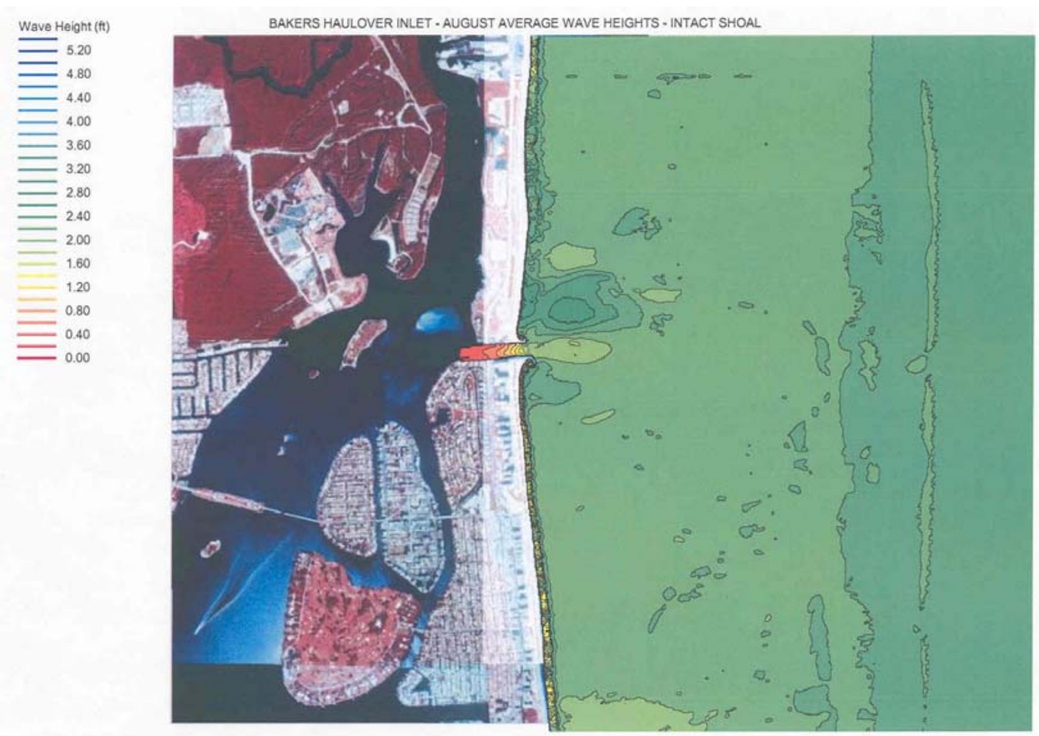


Figure 20(h). Wave Refraction Diagram – Average August Conditions
(Hs = 2.3 ft; T=6 sec; dir = 85 degrees)

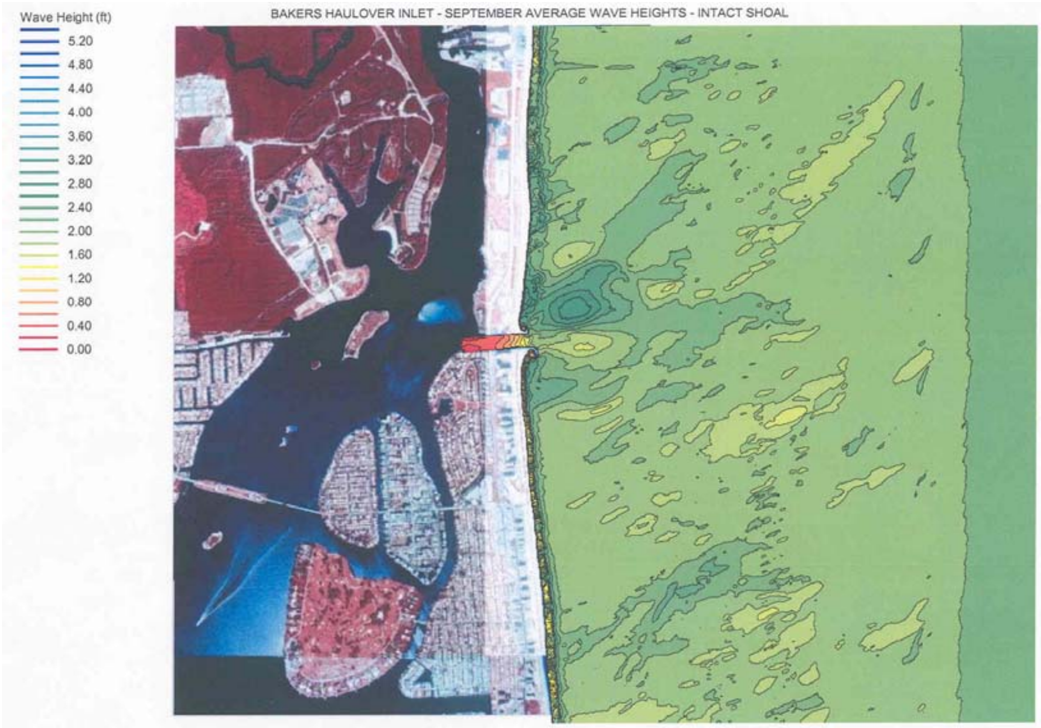


Figure 20(i). Wave Refraction Diagram – Average September Conditions
(Hs = 2.3 ft; T=10 sec; dir = 45 degrees)

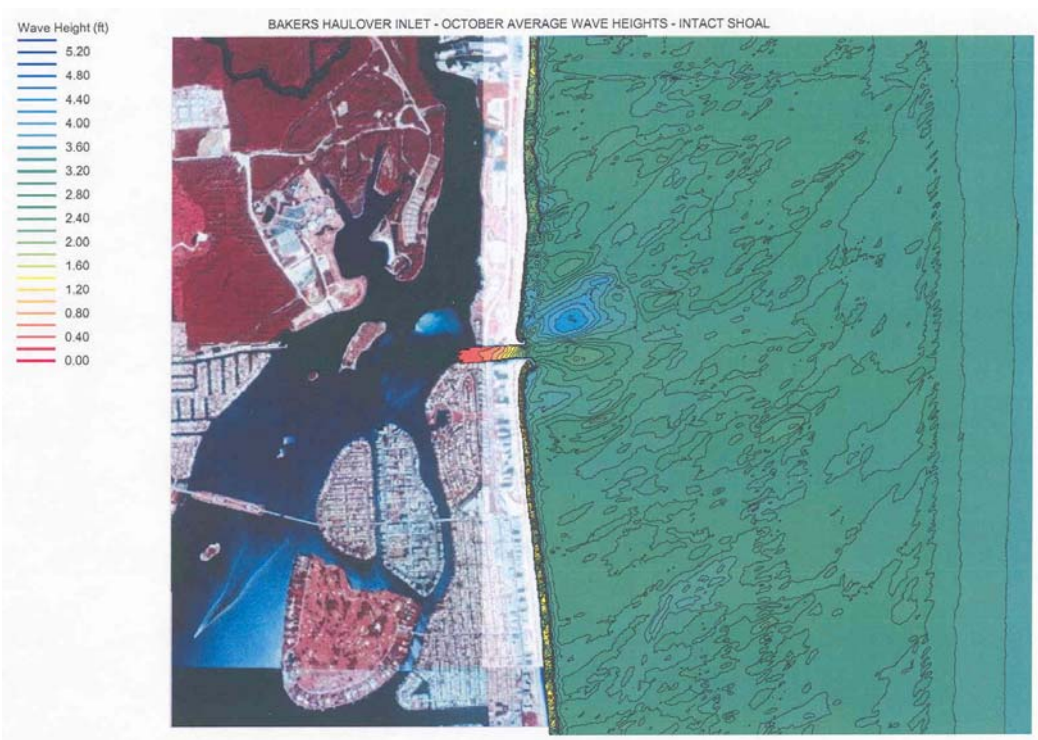


Figure 20(j). Wave Refraction Diagram – Average October Conditions
(Hs = 3.6 ft; T=9 sec; dir = 45 degrees)

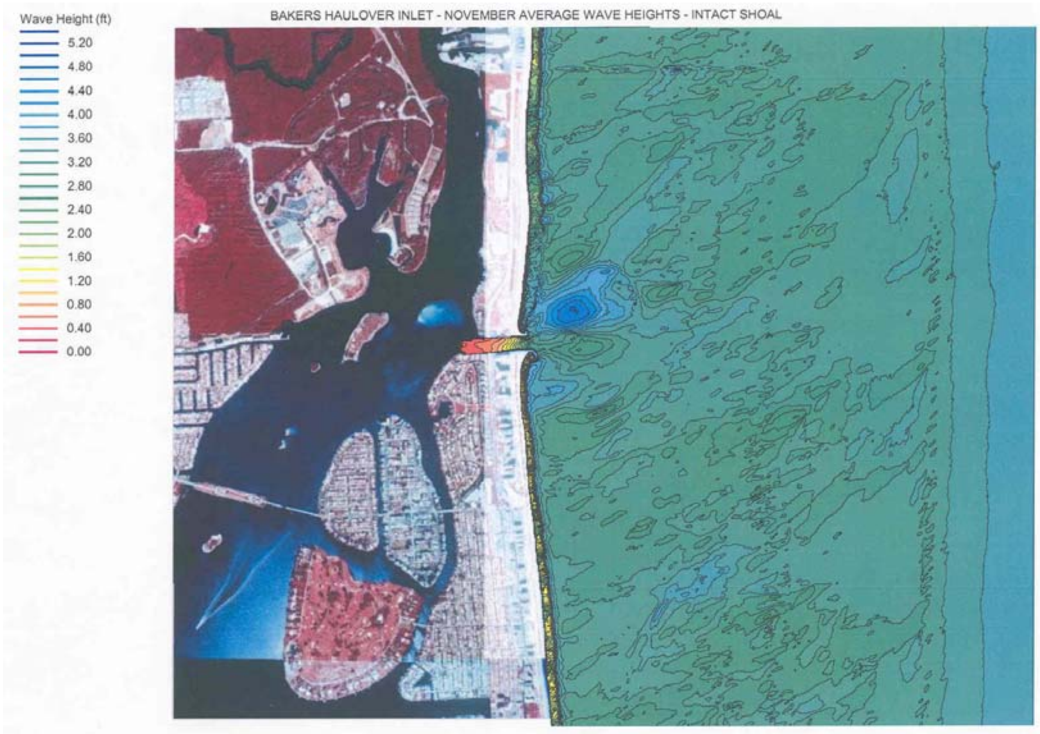


Figure 20(k). Wave Refraction Diagram – Average November Conditions
(Hs = 3.9 ft; T=9 sec; dir = 45 degrees)

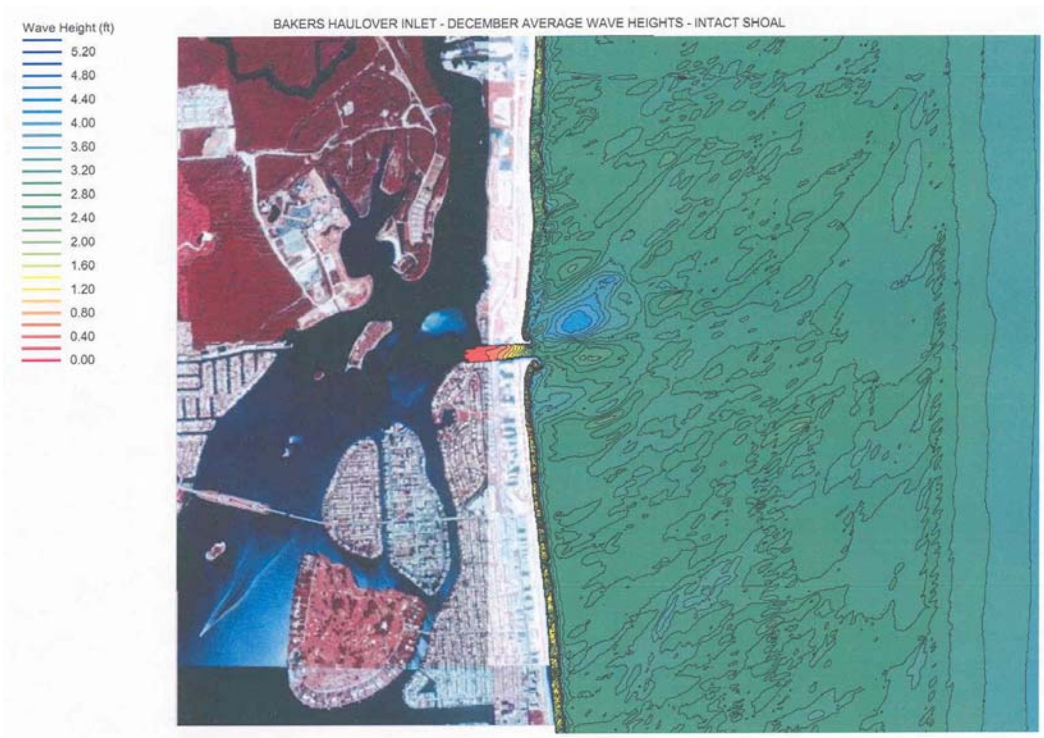


Figure 20(l). Wave Refraction Diagram – Average December Conditions
(Hs = 3.9 ft; T=9 sec; dir = 35 degrees)

The most important factor concerning wave energy focusing is not necessarily the absolute magnitude of the wave heights along the shoreline, but rather the wave height gradient in the alongshore direction. When areas of relatively higher wave energy exist, such as in the vicinity of Bakers Haulover Inlet, the tendency is for sediment to erode much more rapidly from these high-energy areas, and sediment transport often occurs from areas of higher wave energy towards areas of lower wave energy. An examination of STWAVE output data reveals that wave heights along the northern 2,000 feet vary from 11 to 33 percent higher than corresponding wave heights along the southern Bal Harbour shoreline for each of the average monthly conditions. The degree of wave energy amplification generally increases with incident wave period. The model-predicted results generally agree with wave height observations made at the project area under a variety of conditions, and verify that wave energy focusing along the northern Bal Harbour shoreline is responsible for at least a portion of the shoreline erosion experienced along that area.

In order to further examine this effect, several different sets of output from the STWAVE model are provided in this report. Close-up displays of model output are included which provide more detail on wave energy focusing along the Bal Harbour shoreline. A second series of figures show the vector output from these same runs; these figures show how wave directions are modified near the ebb shoal. Other sets of STWAVE runs were performed to examine the effects of tidal currents on wave refraction patterns near the inlet, and to determine the changes to existing refraction patterns as a result of modifications to the ebb shoal. Another series of STWAVE output shows the effects of the “northeaster” storms described previously. Due to space limitations the output from these additional model simulations will be presented in Appendix C of this report. The output from these various STWAVE simulations will now be discussed.

The first series of figures in Appendix C (figures C-1(a) through C-1(l)) show a close-up view of the same model output presented in figure 20(a) – 20(l), but with more detail shown along the Bal Harbour shoreline. The display extends from about 2,000 feet north of Bakers Haulover Inlet southward to the southern boundary of Bal Harbour. This display better captures the areas of wave energy focusing landward of the ebb shoal, and still includes the entire length of the Bal Harbour study area. From these figures it is seen that under existing conditions, average wave heights along the northern 2,000 feet of Bal Harbour (and generally the southern 2,000 feet of Haulover Park) are noticeably higher than along the adjacent shorelines further from the inlet. Analysis of STWAVE output data indicates that wave heights along the northern 2,000 feet of Bal Harbour range from about 11 to 33 percent higher than along the adjacent beaches, for a variety of incident wave conditions.

The second series of figures in Appendix C contains vector output from the same series of runs as shown in figure 20 of the main report, and figures from series C-1 of Appendix C. Arrows on the displays in figures C-2(a) through C-2(l) of Appendix C show how the directions of incoming waves are modified by refraction around the ebb shoal. Convergences of these wave direction vectors result in increased wave heights in the vicinity of the ebb shoal and along the shoreline near the inlet, as discussed previously. The focusing of wave energy along the northern Bal Harbour shoreline is most noticeable in the fall/winter/spring months, when incident wave energy is from the northerly directions. An examination of the vector plots in figures C-2(a) through C-2(l) show that this convergence is not as noticeable during the months of May, June, July, and August, when wave energy originates from the southerly directions. The smaller wave amplitudes and shorter wave periods which are commonly observed through the summer months in south Florida may also be a factor in the reduced degree of wave convergence near the inlet. Note also that despite the relative lack of directional convergence, wave energy focusing still occurs during the summer months as shown in figures C-2. This focusing is assumed to be due to wave shoaling along the edges of the ebb shoal.

The vector plots are also useful in locating the nodal point where the direction of longshore sediment transport reverses due to wave refraction. Additional STWAVE runs were performed to further refine the location of the nodal point under a variety of incident wave conditions. Based on these results, this point is located from about 600 to 1,100 feet south of the Bakers Haulover Inlet south jetty. Some general trends emerged from the STWAVE analysis relating the position of the nodal point to the incident wave conditions. First, the nodal point shifts closer to the inlet with increasing wave period. Short wave periods (3 – 5 seconds) result in a nodal point from 750 to 1,100 feet south of the south jetty, while longer wave periods (8 - 10 seconds) result in a nodal point only 600 to 850 feet south of the jetty. Also, incident wave direction has an effect on the location of the nodal point. The point tends to shift further southward as the approach angle of the incident waves becomes more shore-normal. For example, for a 6-second wave period, the nodal point is located 750 feet south of the jetty for extreme northerly angles of incidence, but this distance increases to about 1,000 feet as the angle of approach rotates toward shore-normal. For all southerly angles of wave approach, sediment transport is in a northerly direction throughout the study region, and no noticeable nodal point exists.

Tidal currents passing through Bakers Haulover Inlet can have both direct and indirect effects on sediment transport along the study area shorelines. The direct effects are produced by the flow of tidal currents along the shoreline, which can induce sediment transport. Indirect effects are due to the influence of tidal currents on wave refraction patterns, which can alter existing wave-induced sediment transport patterns. Flood tidal currents flow from the ocean, through the inlet, and into Biscayne Bay. Ebb currents flow in the opposite direction.

Due to the short lengths of both jetties at Bakers Haulover Inlet, flood tidal currents can flow directly along the study area shorelines toward the inlet. Although flood currents velocities directly along the shore are low, they still tend to increase the longshore current velocities and therefore directly influence the transport of sediment toward the inlet. Both ebb and flood tidal currents passing through Bakers Haulover Inlet can have a strong impact on wave refraction patterns in the study area. The third set of figures in Appendix C shows the results of flood tidal currents on wave refraction throughout the study area. As seen in figures C-3(a) through C-3(l), wave energy focusing is reduced slightly in the vicinity of the ebb shoal and the adjacent shorelines under average spring flood tide conditions. This is a normal effect of flood tidal currents on wave refraction patterns near tidal inlets : incoming waves in the vicinity of the inlet are accelerated toward the inlet via the flood current, thereby ‘fanning out’ in shape and dissipating wave energy.

A comparison of figures C-2 (no currents) with C-3 (spring flood currents), shows that the degree of wave energy focusing along northern Bal Harbour is about the same, but the areal extent of the focusing is reduced under flood tidal conditions in all cases. This effect is most noticeable along the southern lobe of the ebb shoal, located about 1,000 to 2,000 feet south of Bakers Haulover Inlet. Since this simulation was performed using the higher velocities of spring flood tide conditions, the effects of average flood tidal velocities on wave refraction should be less. The only noticeable effect that flood tidal currents may have on the study area is to allow a slight tendency toward reduced erosion (and possible minor accretion) near the inlet during the times of strong flood tides. This effect is considered to be minor according to the refraction diagrams in figures C-3, and since maximum flood tide velocities occur over a relatively short time interval.

The effects of ebb tidal currents on wave refraction are generally much greater than the effects of flood tidal currents. Ebb tidal currents tend to increase transport toward the inlet by two processes. First, strong ebb flows through the inlet tend to create large vortexes (or eddies) which can reach the shoreline. South of the inlet these large eddies rotate clockwise on the ebb flow, and when in close proximity to the beach can transport sediment northward toward the inlet (north of the inlet the ebb flow generates counterclockwise eddies, which also direct sediment toward the inlet). Secondly, ebb tidal currents flowing into the open ocean refract incoming waves to a higher degree, causing waves to strike the shoreline near the inlet at a much steeper angle, directed toward the inlet. Ebb currents affect refraction by decelerating waves as they approach the inlet and encounter the opposing current. Wave velocity is reduced in the vicinity of the inlet, resulting in increased refraction of the wave front towards the throat of the inlet. This inward focusing of the wave front results in increased wave energy in the vicinity of the inlet, and a higher angle of wave attack along the shoreline near the inlet. Waves can be refracted to such a degree during strong ebb flows that they are directed towards the inlet on both sides of the channel, increasing transport of beach sediment into the inlet channel from both of the adjacent beaches.

The fourth series of figures in Appendix C shows the effects of ebb tidal currents on wave refraction throughout the study area. As seen in figures C-4(a) through C-4(l), wave energy focusing is increased substantially, but in most cases only in the vicinity of the south jetty. During the fall/winter/spring months when wave energy originates from the northerly directions this wave energy increase occurs over the greatest area, extending from the inlet up to about 1,000 feet south of the south jetty. An examination of STWAVE output data shows that ebb tidal currents increase wave heights in the vicinity of the south jetty at Bakers Haulover Inlet by 31 to 53 percent relative to average wave heights along the shoreline, during the months with northerly incident wave directions. During the summer months (east/southeast incident wave directions) wave heights increase by up to 162 percent relative to average wave heights along the coast, but under these conditions wave energy focusing due to ebb currents occurs only in the inlet channel, so littoral processes remain largely unaffected. It is reasonable to expect that the degree of wave energy focusing near the inlet due to ebb flow will vary throughout the tidal cycle, reaching a maximum when current velocities are the highest, as indicated in figures C-4.

The increased wave energy focusing created by the effect of ebb tidal currents may be responsible for a large degree of erosion along the northernmost 1,000 feet of Bal Harbour, as well as increased channel shoaling. This increased wave focusing appears to be particularly true for incident waves from the northerly directions. An unexpected result of the refraction analysis was that a minimal change in refraction patterns is observed north of the inlet under any of the ebb current conditions. This may be due to the shallower depths along the northern edge of the channel, which tends to divert the majority of the ebb tidal flow to the east and southeast.

From the first four series of figures in Appendix C it is seen that the patterns, locations, and magnitudes of wave energy focusing in the vicinity of Bakers Haulover Inlet will vary in a cyclic manner with the tidal flow. The maximum effect of tidal currents on wave refraction patterns is seen at full ebb flow as shown in figures C-4. As the tidal velocities decrease near low tide the region of maximum wave energy focusing will shift southward to near the southern lobe of the ebb shoal as shown in figures 20, C-1, and C-2. When the tidal flow reverses, the pattern of slight wave energy dissipation which accompanies flood tidal currents (figures C-3) will occur. At high tide the no-current refraction patterns return and as the tide ebbs the cycle repeats itself. It is important to note that the diagrams in figures C-3 and C-4 depict spring (maximum) tide conditions, and lower tidal current velocities will have less of an impact on wave energy focusing near the inlet.

The fifth series of STWAVE output diagrams demonstrates the effects of modifying the ebb shoal. This shoal was used as a source of borrow material in the 2003 renourishment of Bal Harbour, and may be used again periodically as a future source of material. The STWAVE refraction diagrams in figures C-5(a) through C-5(l) demonstrate the effects on wave refraction patterns which would be caused by dredging the ebb shoal to the maximum extent allowed by the State of Florida Department of Environmental Protection permit. Figure 21 shows the maximum permitted dredging limits of the ebb shoal as established for the 2003 Bal Harbour renourishment project. As seen in the figure, three distinct dredging areas are permitted along the extent of the shoal, and all three areas are located along the northern lobe of the shoal, which lies entirely north of the inlet channel. The intent of specifying the different zones of allowable dredging was to maintain the general shape of the shoal in order to minimize disruption to sediment bypassing along the shoal and around the inlet.

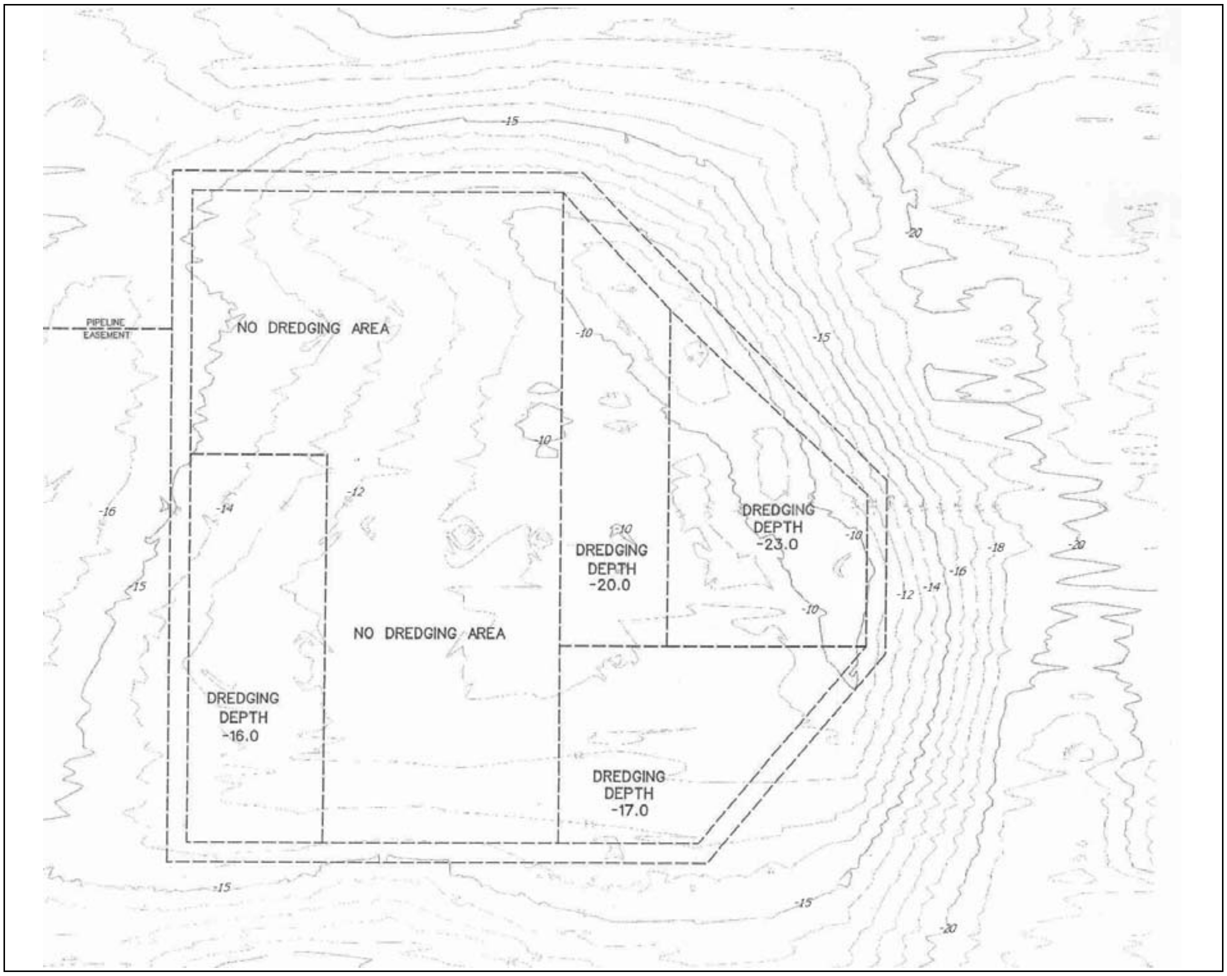


Figure 21. Bakers Haulover Inlet Ebb Shoal, dredging limits.

Figures C-5(a) through C-5(l) can be compared directly with figures 20(a) through 20(l) or figures C-1(a) through C-1(l) to determine the effects of ebb shoal dredging. In most cases it is readily seen that the distribution of wave energy on the ebb shoal is greatly altered, but only a very minor increase in wave energy occurs along the shoreline north of the inlet in some specific cases, due to the reduced sheltering effects of the shoal. The changes in wave energy distribution along the shoreline as a result of maximum dredging of the ebb shoal are barely discernable north of the inlet, and no visible changes to the existing pattern of wave energy focusing can be seen south of the inlet along the Bal Harbour shoreline. Given the low magnitude of changes in wave energy distribution along the shoreline, it appears that no significant effects on littoral process will result from the periodic dredging of the Baker Haulover Inlet ebb shoal to the limits shown in figure 21. Based on transport rates presented previously in the sediment budget, this borrow area should refill at the rate of approximately 60,000 cy/yr. Based on a typical renourishment volume of 200,000 to 300,000 cubic yards the fully-excavated borrow area should completely refill every 3.5 to 5 years.

The sixth and final series of STWAVE output diagrams in Appendix C pertains to the storm wave input previously described. In order to determine the impacts of large (but still relatively frequent) storms on the project area, two representative ‘northeaster’ storm were input into the STWAVE refraction model. Parameters from both storms were extracted from the WIS database as described earlier : the waves representing storm #1 occurred on 3 January 1993, and the waves representing storm #2 occurred on 21 January 1993. The incident deepwater waves for storm #1 consist of 7.0-foot, 7-second waves, originating from 82 degrees clockwise of due north. The incident deepwater waves for storm #2 consist of 10.8-foot, 9-second waves, originating from 69 degrees clockwise of due north.

Results of these model runs are shown in figures C-6(a) through C6(f). Three refraction diagrams are provided for each storm event, consisting of one diagram each representing storm waves superimposed on the no tidal current condition, on spring flood currents, and on spring ebb tidal currents. As can seen in each of these figures, large storm waves break well offshore of the shoreline along the inshore reef line, particularly for the larger waves of storm #2. This reef line is shown in the lidar bathymetry map presented in figure 18. The STWAVE output shows the considerable drop in wave amplitudes along this nearshore reef due to wave breaking along this reef line.

In spite of the considerable reduction of the higher wave amplitudes along this reef, a minor degree of wave energy focusing still occurs along the northern Bal Harbour shoreline due to the refraction of the broken waves by the ebb shoal system. Note that the ebb shoal provides some degree of sheltering along the shoreline of southern Haulover Park and in the inlet region due to extensive wave breaking along the outer reaches of the shoal. This sheltering is particularly obvious for storm #2.

The minor degree of wave energy focusing along northern Bal Harbour is observed in both storm events, for all tidal conditions. The difference in alongshore wave energy distribution in the STWAVE output is considerably less that was observed under the “average” condition simulations in figures C-1 through C-5. Since the “average” conditions occur much more frequently and represent a higher wave energy gradient in the alongshore direction, it is concluded that erosion due to wave energy focusing in the study area is driven more by the ‘normal’ wave climates, and less by storm events.

Refraction Analysis – Conclusions. Several consistent trends are noted from the preceding STWAVE wave refraction analysis. Under a variety of ‘average’ and ‘storm’ wave conditions, wave energy is distributed relatively evenly along the simulated reach of shoreline, with the exception of the region surrounding Bakers Haulover Inlet. The highest degree of wave energy focusing in the project area is noted on the ebb shoal, with lesser areas of focusing along the shorelines north and south of Bakers Haulover Inlet adjacent to the shoal.

Wavelength plays an important role in wave shoaling. Since waves begin to feel the effects of the bottom at a depth of approximately one-half the wavelength, longer-period waves will begin to shoal in deeper water causing these waves to focus into narrow bands further from shore. Shorter period waves are less affected by shoaling and bottom effects throughout most of the nearshore zone. Most shorter period waves in this study area will pass uniformly over the deeper portions of the reef system and will not begin to shoal or refract until they reach the shallower portions of the reef. This effect is demonstrated in figures 20(e) through 20(h) (May through August) which show broader focusing bands that originate closer to shore during the summer months.

Wave height is another factor in wave focusing. While waves with higher amplitudes are more likely to experience energy losses due to whitecapping, wave breaking, and bottom interactions, they also provide more initial energy at the offshore boundary that will be focused as it is propagated. The result is higher wave heights with more energy at focal points along the shoreline. In some cases, however, wave heights are too high relative to water depth and offshore waves break with near uniformity over the outer reef system eliminating or reducing focal points at the shore, as seen in figures C-6. Large storm waves also tend to dissipate a large portion of their energy along the seaward edge of the ebb shoal, and the ebb shoal appears to provide a sheltering effect to the shoreline north of Haulover Inlet due to this effect.

According to a wave refraction analysis performed by the Jacksonville District in 2001, the largest degree of wave energy focusing along the Dade County shoreline occurs on the Bakers Haulover Inlet ebb shoal, where incident wave heights can be increased by up to 50 percent due to refraction and shoaling. The refraction diagrams presented in figure 20 and in Appendix C show the high degree of wave energy focusing that occurs along this shoal under a variety of conditions. In spite of the considerable increase in wave heights along this shoal, the direct effect of these wave height increases on shoreline erosion is minimal due to the distance of the shoal offshore. The center of the ebb shoal is located about 2,500 feet offshore, and wave energy dissipated this far from the shoreline has little direct effect on nearshore littoral processes.

Even though the majority of the wave energy focusing occurs well offshore along the crest of the ebb shoal, the shoal still plays an important part in the wave energy distribution along the coastline. Waves refracted around the ebb shoal continue to propagate landward, and appear to be re-focused on the adjacent shorelines as they pass landward of the shoal. Two areas of wave energy focusing occur under almost every incident wave condition : the reach of shoreline extending up to 2,000 feet north of the Bakers Haulover Inlet north jetty, and the reach of shoreline extending 2,000 feet south of the south jetty. The latter reach is of particular interest in this study.

As shown in figures 20(a)- 20(l), wave energy is focused along the shoreline north and south of Bakers Haulover Inlet, extending outward for a distance of about 2,000 feet. The increased wave heights north of the inlet have no direct impact in the Bal Harbour project area, but may have the indirect effect of transporting additional sediment into the inlet, where a portion of the sediment is transported into the ebb shoal via tidal currents. An increase in size of the ebb shoal could tend to increase the wave energy focusing along the adjacent shorelines.

A more direct effect of the focusing is seen in figure 20 and in figures C-1 through C-5 in Appendix C. Waves passing landward of the ebb shoal are focused along the 2,000-foot reach of northernmost Bal Harbour. Depending on the incident wave condition, refracted wave heights along the northern 2,000 feet of Bal Harbour may be 11 to 33 percent greater than along the shorelines further to the south. The area of wave focusing predicted in the STWAVE numerical model is virtually identical to the reach of shoreline which has experienced the most severe erosion since the project was constructed in 1975.

The refraction of incident waves around the ebb shoal/inlet complex also results in the creation of a nodal point south of the inlet in an area where wave directions diverge due to refraction. This nodal point appears in the STWAVE simulations for all incident wave conditions from the northerly and easterly directions. For all northerly and easterly wave conditions the location of this nodal point varies from about 600 to 1,100 feet south of the Bakers Haulover Inlet south jetty, depending primarily on incident wave period and direction. Under these conditions, sediment north of the nodal point is transported northward; sediment south of the nodal point is transported southward. For waves originating from southerly directions, littoral transport is directed toward the north throughout the entire region and no nodal point exists.

The most severe area of observed erosion along Bal Harbour extends along the northern 2,000 feet of the project shoreline. As indicated in the preceding STWAVE analysis, three processes contribute to this erosion. First, the net annual littoral transport direction is southward, and the blockage of southward-directed sediment caused by the inlet channel/ jetty complex results in erosion immediately south of the inlet. Second, an area of wave energy focusing due to refraction occurs along the northern 2,000 feet of the Bal Harbour shoreline. Sediment tends to be transported from this region toward adjacent areas of lower wave energy. Third, a nodal point in sediment transport direction exists due to the effects of wave refraction. This nodal point is generally located 600 to 1,100 feet south of the Bakers Haulover Inlet south jetty, near the midpoint of the region of most severe erosion. During periods of northerly and easterly waves (the predominant wave-energy directions) sediment north of the nodal point is transported northward toward the inlet, while sediment south of the nodal point is transported southward.

According to the preceding STWAVE analysis, the erosive effects along the northern reach of Bal Harbour will become somewhat more severe during periods of ebb tidal flow, and slightly less severe during flood tidal flow due to the effect of tidal currents on wave refraction. No noticeable impacts to existing wave refraction patterns occur as a result of excavating the Bakers Haulover Inlet ebb shoal to it’s permitted dredging limits. The relative magnitude of the erosion attributable to each of the processes described above is difficult to determine. In order to gain further insight into these processes the GENESIS shoreline change model was used.

GENESIS Shoreline Change Model.

The shoreline change model GENESIS (GENEralized Model for the SImulation of Shoreline Change) was used to calculate sediment movement along the shoreline within the study area. GENESIS is a finite-element, empirically based model driven by time-sequenced wave data. This model simulates long-term shoreline response to wave attack, and allows the input of various types of coastal structures along the shoreline. The GENESIS model will be used in this report to better define trends in nearshore sediment movement, and to determine the relative merits of alternative plans of improvement along the study area.

The general procedure for executing the GENESIS model involves gathering data on shoreline positions along the study area at specific points in time, and data on the wave climate during the interval between the various shoreline surveys. Other data such as berm height, depth of closure, median sediment grain size, the location and position of structures, and boundary conditions are also included as model input. The model is calibrated by using two known sets of shoreline positions along the length of the study area and the corresponding wave data for the interval between the shoreline surveys. Adjustments to the model can be made by varying the values of two model calibration coefficients, designated as K_1 and K_2. When the model-predicted shoreline matches the actual surveyed shoreline, the model is considered to be calibrated correctly. At this point a second set of surveys and wave data are run to verify the model results. Upon successful verification the GENESIS shoreline change model is ready for use, and various alternative plans of improvement can be tested using the model.

The first step in running the GENESIS model is setting up the model grid. Figure 19 shows the selected location of the GENESIS grid along the Bal Harbour shoreline. The GENESIS baseline extends along the shoreline, and is subdivided into a number of computational cells. The number of cells is limited mainly by the capacity of the computer used for the simulation, but for this study a limit of 300 computational cells was used, resulting in a grid cell spacing of 25 feet along the entire length of Bal Harbour and into northern Surfside.

As seen in figure 19, the GENESIS grid is nested inside the STWAVE computational grid. Due to the complex bathymetry offshore of the project area, STWAVE was used to refract incident deepwater waves to a nearshore stationing line for input into the GENESIS model. By running the input wave files through STWAVE prior to GENESIS input, more realistic results were obtained by accounting for the refractive effects of the complex bathymetry of the area, including the Bakers Haulover Inlet ebb shoal.

Since the primary goals of GENESIS modeling were to more accurately define nearshore sediment movement and test alternative plans of improvement for the Bal Harbour study, input wave events with the highest frequency of occurrence were selected for numerical shoreline simulation. The WIS wave database contains the longest period of record available for the South Florida area, extending from 1956 through 1995. The database from WIS Station 9 was used in the STWAVE analysis presented above, and will also be used in the GENESIS shoreline simulation modeling. In the STWAVE analysis statistical methods were applied to the WIS data to develop one average wave condition for each month of the year.

For the GENESIS simulation a time-series of events is required, and the WIS database was analyzed to develop this input database. Each of the forty years of wave data in the WIS database contain 2,920 wave events, at 3-hour intervals. Since the entire forty-year record consists of 116,800 individual wave events, shoreline modeling using the complete record is not practical, and an abbreviated input wave data file was required.

Due to the seasonal variations of the south Florida wave climate it was determined that the minimum duration of the input time-series would be one year, in order to capture the seasonal variations between summer and winter months. An "average" wave-energy year was desired, in order to represent the most frequently-occurring events, so the entire 40-year WIS record was analyzed to select the most representative average year. The program SEDTRAN was used to calculate relative sediment transport potentials. For each year of WIS data, SEDTRAN was used to calculate the gross, net, north–directed, and south-directed sediment transport. The 40-year averages were calculated for each category of transport, and the year closest to the average values was selected for use as the "average" wave-energy input file for GENESIS modeling. This selected average wave-energy year was 1993.

The GENESIS program was run within the framework of the NEMOS software package (version 2.01G), which also contains STWAVE and several wave database management files. The WIS database includes two series of wave records : a primary wave component (corresponding to swell) and a secondary wave component (corresponding to locally-generated seas). These components are combined into a permutations file which is run through STWAVE to obtain wave transformation data along a selected nearshore reference line for GENESIS input. This reference line was chosen to be along the 9-foot depth contour extending along the Bal Harbour shoreline. The 9-foot contour was chosen because the majority of wave events from the WIS file will not have broken seaward of this contour. GENESIS reads the input wave data at this nearshore reference line and then uses an internal refraction routine to shoal the refracted waves to breaking depths and calculate sediment transport potential.

Other GENESIS input data includes median sediment grain size of the existing beach (0.35 mm), berm elevation (+9 feet mlw), and depth of closure (-17 feet, mlw). Boundary conditions are specified at each end of the model grid. At the north end of the grid the Bakers Haulover Inlet south jetty was modeled as a gated boundary, meaning in this case that sediment can flow around the jetty and into the inlet, but little sediment can flow from the inlet onto the model grid. The south end of the model grid lies along an open reach of shoreline in northern Surfside, at a location where the shoreline recession is known. From the mean high water line position change analysis presented previously in this report, a recession value of –2.8 feet per year is calculated. This moving boundary condition was specified for the southern end of the GENESIS grid.

Finally, the existing structures were entered into the model domain. The Bakers Haulover Inlet south jetty and five existing king pile groins were each entered into the input file. The correct lengths of each structure were obtained from aerial photos and the 2002 lidar survey.

The permeability of the south jetty was set at 0 percent, since the core of the structure is essentially sand-tight. The five existing king pile groins provide minimal disruption to sediment flow along the beach, so the permeability of each structure was set to 90 percent.

The two shorelines chosen for model calibration were surveyed in 1990 and 1996. These surveys correspond to shortly after the 1990 Bal Harbour beach renourishment project, and about 2 years prior to the 1998 Bal Harbour renourishment project. No beach fill placements or other modifications occurred along Bal Harbour during this simulation period. The average volume change from 1990-2002 was –55,620 cy/yr; this was used as a target value in GENESIS calibration, along with a target littoral transport rate of about 48,000 cy/yr southbound, near the south end of the project. Several variations of the volume and distribution of the bypassing rate (19,000 cy/yr, from CSI sediment budget) were simulated. Optimum agreement with observed shoreline response was noted with approximately 60 percent of the bypass volume entering the model domain along a 1,000-foot reach of the shoreline centered on groin #1, and the remaining 40 percent being distributed along the remaining length of the model grid. This distribution corresponds very closely to the visual extent of the ebb shoal.

Once the model was successfully calibrated, a series of verification runs was performed using a different set of shorelines. The interval from 1996 through 1998 (1998 pre-fill) was chosen because, like the calibration simulation interval, it contained no beachfills or other construction activity. The same average volumetric change rates and littoral transport rates were used as target values during the verification phase. The calibration and verification were completed and the two GENESIS calibration coefficients were determined as K_1 = 0.15, and K_2 = 0.10. The GENESIS model was now ready for use in refining the project area sediment budget and simulating various alternative plans of improvement.

Updated Sediment Budget.

Portions of the Bal Harbour sediment budget were laid out in the preceding discussion of littoral transport rates throughout the study area. Specifically, the rate of sediment transport southward into Bal Harbour via inlet bypassing was calculated at 19,000 cy/yr, and sediment transport at the south city limit of Bal Harbour was estimated at 48,000 cy/yr, southward. The average annual volume change along the Bal Harbour shoreline was measured at –55,620 cy/yr, based on an analysis of surveys from 1990 to 2002.

From examinations of aerial photographs and lidar bathymetry of the ebb shoal area it is obvious that the ebb shoal extends continuously around the inlet, except for the low area in the shoal where the channel crosses. The region where the southern lobe of the shoal approaches the Bal Harbour shoreline is marked by a slight increase in berm width on the shoreline, as shown near the center of Bal Harbour (near groin #2) in the aerial photographs in figures 2 and 15. These figures also show a continuous sand bridge between the shoal and the nearshore bar system in the same region, and based on the presence of these features the southward flow of sediment along the ebb shoal and onto Bal Harbour's beaches is evident.

Based on the width of the perturbation in the shoreline near the southern terminus of the ebb shoal, it does not appear likely that the entire annual bypass volume of 19,000 cy/yr is re-deposited along this narrow region of the Bal Harbour shoreline. Based on results from the calibration/verification phase of the modeling, the distribution of this sediment inflow was determined to be : 60 percent (11,400 cy/yr) along a 1,000-foot reach of shoreline centered on groin #2, and 40 percent (7,600 cy/yr) along the remaining reach of the model grid to the south. It should be noted that the GENESIS grid extends approximately one-half mile south of the Bal Harbour south city limit, so approximately one-half of the 7,600 cy/yr (= 3,800 cy/yr) is transported onshore south of the Bal Harbour study area. Therefore, of the 19,000 cy/yr bypassed southward around Bakers Haulover Inlet, 15,200 cy/yr is deposited within the littoral system of Bal Harbour.

The main mode of bypassing appears to be directly along the main body of the shoal via wave action. This material would be transported onshore in the vicinity of groin #2 as described above. The material bypassed onshore further south appears to be transported by a more indirect mechanism. Particularly in figure 2, large sand formations can be seen breaking off from the southern lobe of the ebb shoal. Most of these formations have the appearance of waves or ridges, and the larger ridges are also seen in the lidar bathymetry. These sand waves contain large volumes of sediment, and several waves can be seen in what appears to be a southward and onshore–directed progression from the main body of the ebb shoal. As seen in the preceding wave refraction analysis, the direction of the predominant wave energy in the area indicates that these deposits should be transported in a southwesterly direction, eventually depositing the material onshore further to the south.

Based on the preceding data the updated sediment budget is calculated as follows. The measured average annual net volume change along the Bal Harbour shoreline is –55,600 cy/yr. Accounting for the annual deposition of the bypass volume of 15,200 cy/yr, the gross volumetric change becomes (55,600 + 15,200) = 70,800 cy/yr. From GENESIS output data using the existing shoreline configuration, the longshore transport northward from the nodal point constitutes 33 percent of erosional losses; longshore transport southward from the nodal point constitutes 67 percent of erosional losses. Applying this ratio to the calculated gross transport rate of 70,800 cy/yr results in volumes of 47,400 cy/yr lost from the south end of the project, and 23,400 cy/yr lost from the north end of the project. Note that the GENESIS-predicted loss rate from the south end of Bal Harbour (47,400 cy/yr) agrees almost exactly with the previously calculated net longshore transport rate of 48,000 cy/yr.

Observations at the project site corroborate this sediment budget. The onshore transport of sediment from the ebb shoal is marked by a slight widening of the shoreline at the southern lobe of the shoal, which is easily seen in most aerial photographs of the Bal Harbour shoreline. The area of wave energy focusing and accelerated erosion along the northern 2,000 feet +/- of Bal Harbour is also observable in aerial and ground-level photos, and in the surveys. The northward transport of sediment along the northern reach of the shoreline is evidenced by a permanent filet feature immediately south of the south jetty. Finally, the relative ineffectiveness of the existing groin system is demonstrated by the lack of updrift buildup/downdrift erosion adjacent to the groins, as observed in surveys and aerial photos.

PLAN FORMULATION

Goals of Proposed Improvements.

From the preceding analyses it is seen that accelerated erosion occurs along the Bal Harbour shoreline due to several processes. Wave energy is focused along the northern 2,000 +/- feet of Bal Harbour shoreline due to refraction around the Bakers Haulover Inlet ebb shoal under most incident wave conditions. Shoreline erosion is accelerated in this zone of increased wave energy. Furthermore, refraction redirects these waves to the degree that sediment transport is to the north along the northern 1,000 +/- feet of Bal Harbour under a wide variety of incident wave conditions. The net sediment transport along the remaining reach of Bal Harbour and the adjacent shorelines is southward. Since Bakers Haulover Inlet creates a littoral barrier to this southward sediment transport, erosion is further accelerated immediately south of the inlet. This downdrift erosion is typically the most severe during the fall, winter, and spring months when southerly littoral transport occurs. During the summer months northward littoral transport occurs, causing some minor accretion along the northern Bal Harbour shoreline.

The goal of the proposed improvements is to reduce erosion rates along the Bal Harbour shoreline, particularly along the northern 2,000 +/- feet where wave energy focusing (and associated erosion) is the most severe. A nodal point exists during most of the year when southerly transport occurs (fall, winter and spring months). This nodal point is located about 600 to 1,100 feet south of the south jetty, (near groin 1) depending primarily on incident wave period and direction. Material north of the nodal point is transported northward into the channel and is essentially lost from the system. The reduction of this northward flow of sediment will be an objective of all alternative plans of improvement. Reducing the flow of sediment into the inlet will have the added benefit of reducing shoaling of the Federal navigation channel and the associated maintenance dredging of that project.

The sediment deficit induced by the littoral blockage created by the Bakers Haulover Inlet is another component of the erosion observed along the Bal Harbour shoreline. Another objective of proposed improvements is to reduce long-term losses of sediment from the Bal Harbour shoreline without inducing downdrift erosion along the shorelines to the south. Sediment transport is predominantly to the south (south of the nodal point) and measures should be formulated to improve sand retention along this portion of the project.

The ebb shoal at Bakers Haulover Inlet provides a pathway for natural transport of sediment around the inlet. A primary objective during plan formulation is to allow maximum natural bypassing of sediment to continue along this shoal. Any structural plan which significantly interferes with the predominantly southward transport of sediment along this shoal will likely result in increased erosion south of the inlet, corresponding to higher renourishment rates and associated project costs. The ebb shoal re-connects to the Bal Harbour shoreline in the vicinity of groin 2, and sediment transport from the shoal to the beach appears to occur along most of the remaining reach of Bal Harbour to the south.

Alternative Plans Of Improvement.
Several alternative plans were developed which address the needs outlined in the previous section. Traditional methods of shore protection will be examined including the use of rubble-mound groins, breakwaters, and revetments. “Innovative” technologies will be examined as well; these alternatives consist primarily of variations of the traditional structures using different materials and/or methods of construction. Different beach fill designs will also be examined, both in traditional and in innovative configurations. The “no-action” plan will be used as the baseline condition against which these alternatives will be evaluated. Alternatives will be designated according to the following groupings : No-Action plans will be designated “NA- “, Structural plans will be designated “S- “, Beach fill configurations will be designated “B- “, and Innovative plans will be designated “I- “.

No-Action Plans. Two plans will be considered under this category, as described below.

Alternative NA–1. The Primary No-Action Plan. The existing groin field would be left in a deteriorated condition, exactly as it currently exists. The project would continue to be renourished on an “as-needed” basis, using the existing 240-foot wide beach fill template.

Alternative NA-2. Modified No-Action Plan. Similar to alternative NA-1, but all five of the existing groins would be removed entirely. Renourishment would continue to be provided on an “as-needed” basis using the 240-foot template.

Structural Alternatives. The structures described below would be constructed in the “traditional” manner, using rubble-mound construction.

Alternative S-1. Rehabilitation of Existing Groins. The five existing king pile groins would be repaired, or sections of the structure replaced as necessary. Each groin would be reconstructed to its original length. The porosity of these structures can be adjusted by adding or removing the horizontal panels between the king piles. Groins of various types have long been used to reduce the longshore movement of sand, especially in areas with limited sediment transport reversals.

Alternative S-2. Construction of New Groin Field. The five existing king pile groins would be removed and new rubble-mound groins constructed in their place. The new structures could be of different lengths and/or at different locations than the original structures in order to optimize performance. Additional groins could be added (or removed) if needed.

Alternative S-3. Construction of T- (or “Tuned-”) Groin Field. This option would be similar to alternative S-2. The existing king pile groins would be removed and new rubble-mound groins constructed, either in the footprint of the original groins or in new locations. The only difference between plans S-2 and S-3 is the addition of the “T”- segments on the seaward end of some (or all) groins. The T-segments on the end of groins can increase performance of the groin field by more effectively holding material between the structures, and by reducing rip currents and subsequent losses of fill into deep water. The T-head groin design has been used in similar locations to contain downdrift losses near inlets.

Alternative S-4. Construction of Offshore Breakwaters. A series of rubble-mound shore-parallel structures would be constructed offshore along portions of the Bal Harbour shoreline. Breakwaters can reduce erosion by decreasing wave energy along the shoreline in the lee of the structure. This design would be based on similar structures which were constructed at Sunny Isles in 2002.

Alternative S-5. Combination of Rubble Groins and Breakwaters. Any number and configuration of rubble groins and breakwaters could be combined to create an effective solution to the erosion at Bal Harbour. As a general plan, breakwaters would be used near the north end of the project to shield that region from the wave energy focusing predicted by the wave refraction analysis in this report. Groins would be used south of the nodal point to reduce the southward-directed losses of fill material. Various combinations of structures would be devised and tested using the GENESIS numerical shoreline change model.

Alternative S-6. Extension of the Bakers Haulover Inlet South Jetty. The existing southward curve on the south jetty would be removed, and the jetty would be lengthened by up to several hundred feet. The curve could be replaced at the end of the new structure. Extending the jetty would allow a larger volume of material to be impounded at the northern end of Bal Harbour, in the area where erosion is usually the most severe. The curve at the seaward end of the existing jetty has proven useful at reducing the formation of large vortices (especially on ebb tide) which can adversely affect the shoreline, and at reducing scouring near the seaward tip of the structure due to tidal currents.

Alternative S-7. Construction of Sand Bypassing Facility. A fixed bypassing facility would be constructed, similar to the existing facility at Lake Worth Inlet in Palm Beach County. The plant would pump material from an impoundment area on the north side of Bakers Haulover Inlet, via pipeline to a pumpout area along Bal Harbour. Such a facility would increase the rate of bypassing around the inlet and would decrease erosional losses along the Bal Harbour shoreline due to the disruption of littoral transport created by the inlet. This plan could include extension of the north jetty to increase impoundment capacity.

Alternative S-8. Close Bakers Haulover Inlet. The two existing ocean jetties would be dismantled, and stone from the jetties and fill from the ebb shoal would be used to fill in the inlet throat. Natural littoral transport along the shoreline would be restored and localized erosion on both sides of the inlet would be reduced or eliminated.

Beach Fill Alternatives. The conventional offshore sources of borrow material have been essentially depleted along the Dade County coastline. Each of the following beach fill alternatives assumes that borrow material can be obtained from any number of non-conventional sources, which will be explored in greater detail in other sections of this report.

Alternative B-1. Construction of "Historic" Beach Fill. This option would result in the reconstruction of the original construction berm, which consists of a template 240 feet wide at elevation +9.0 feet mlw, with a front slope of 1v:11h.

Alternative B-2. Construction of Beach Fill of Altered Dimensions. This option would be similar to alternative B-1, except that the fill dimensions could be modified to optimize the performance of the fill. For example, a narrower berm may be specified in areas where erosional losses are lower, so that the berm erodes at a more uniform rate along the length of the project.

Alternative B-3. Construction of Feeder Beach. Selected portions of the project area could be overfilled to allow sediment to migrate into erosive regions of the project. Specifically, sediment would be placed in the region where the southern lobe of the ebb shoal connects to the Bal Harbour shoreline, to allow additional fill to migrate northward into the erosive area.

Alternative B-4. Construction of Nearshore Berm. Beach fill would be placed in the nearshore region, versus direct placement on the beach. This fill configuration would serve two purposes : to provide a source of sediment for onshore migration, and to act as a low-crested breakwater, reducing wave heights along the beach.

Alternative B-5. Construction of Perched Beach. Beach fill would be placed along the upland portion of the beach as described in plans B-1 or B-2, but the seaward edge of the fill would be contained by a longshore structure to reduce cross-shore losses. The most promising type of longshore structure would be sand-filled geotextile tubes.

Innovative Technologies. The following alternative plans of improvement rely on the use of innovative technologies. “Innovative technologies” is a broad term which can include entirely new methods of beach preservation, or constructing conventional structures using non-conventional materials. The following alternatives are based on innovative technology designs for shoreline stabilization which have been used successfully at locations similar to the Bal Harbour study area, or which were examined and found to be favorable for use along the Dade County shoreline during the recent evaluation of proposals for the Corps of Engineers’ Section 227 Innovative Technologies program. Under the authority of the Section 227 program “innovative” structures are to be designed and built on an experimental basis at several candidate sites around the U.S. coastline, including at an erosional hotspot located in Miami Beach, approximately 3 miles south of Bal Harbour.

Alternative I-1. Construction of Porous Groins. Several variations of this type of structure were presented as proposals for the Section 227 program. The common element of each proposal was the construction of a porous barrier; the actual composition of the porous element varied substantially among the proposals. The two most promising designs consisted of a stiff, porous PVC mesh (similar to snow fencing), and a series of vertical PVC piles separated by narrow gaps. Each structure would slow the longshore transport of sediment along the shoreline.

Alternative I-2. Artificial Reef Module Breakwater. An offshore breakwater would be constructed similar to the design proposed in alternative S-4, but instead of armor stone the body of the structure would be constructed using concrete artificial reef modules. The structure would have a wider crest to offset the higher porosity (as compared to armor stone) of most types of reef modules. In addition to providing shore protection via reduced wave energy along the shoreline, the structure would also provide recreational and environmental benefits as a nearshore artificial reef. The selected plan for the Miami Beach site in the Section 227 program consisted of a nearshore breakwater constructed of Reefball^{R} artificial reef modules.

Alternative I-3. Beach Mats. The surface of the beach would be covered with geotextile fabric to provide direct protection against erosion by creating a physical barrier between the beach and the ocean waves. Variations of this alternative use large, flat geotextile containers which are filled with beach sand, then placed along the beach face. Both of these measures have been used with some success for riverbank stabilization.

A summary of the alternatives described above is provided in table 18 below.

TABLE 18
SUMMARY OF ALTERNATIVE PLANS

Plan	Description
NA-1	No Action Plan, continued renourishment
NA-2	No Action Plan, continued renourishment, remove groins
S-1	Rehab Existing Groins
S-2	Construct New Groin Field
S-3	Construct T-Head Groin Field
S-4	Offshore Breakwaters
S-5	Combined Structures
S-6	Extend Haulover South Jetty
S-7	Sand Bypass Facility *
S-8	Close Haulover Inlet
B-1	Construct "Historic" Fill Template
B-2	Construct Fill - Altered Dimensions
B-3	Feeder Beach
B-4	Nearshore Berm
B-5	Perched Beach
I-1	Porous Groins
I-2	Reef Module Breakwater
I-3	Beach Mats

Preliminary Evaluation of Alternatives.

Several criteria will be used to evaluate the alternatives listed above, including effectiveness at reducing erosion throughout the project area, minimization of adverse impacts to adjacent areas, environmental compatibility, aesthetics, public safety, local sponsor preference, State of Florida permitting guidelines, longevity/storm survivability, and cost.

Many alternatives can be dismissed based on the criteria above, with no detailed numerical modeling required. During the review of innovative alternatives for the Section 227 study many of those proposals were objectionable to Federal, State, and/or local government officials for various reasons, and were eliminated from further consideration. For example, alternative I-3, which would cover the beach surface with geotextile fabric, was strongly objected to by the U.S. Fish & Wildlife Service, State, and local officials due to environmental reasons (interference with sea turtle nesting), public safety (can not drive emergency/lifeguard vehicles over protected surface), aesthetics, and lack of longevity (UV degradation in the South Florida sun). Alternative I-1 was rejected for similar reasons relating mainly to sea turtle entanglement and aesthetic considerations.

Alternative B-5, Construction of a Perched Beach, was also eliminated at the offset due to environmental and public safety concerns. This alternative would involve the construction of a shore-parallel structure to contain beach fill. The shore-parallel structure would typically be a large geotube or several stacked geotubes. The geotubes would be placed along the length of Bal Harbour in the nearshore region, typically in 3-4 feet of water and backfilled with beach fill. Environmental concerns are based primarily on the potential obstacle that the tubes could present to nesting sea turtles. Concerns have also been raised by permitting agencies in the past as to whether the endangered sea turtles could ingest and choke on the geotextile fabric. Public safety is also a concern, since the perched beach represents a steep dropoff into deeper water in the nearshore zone. Finally, engineering considerations were unfavorable for this alternative, since the greater water depths at the toe of the beach fill would allow increased wave energy to affect the beach, very likely increasing erosion.

Alternatives B-5, I-1, and I-3 have therefore been eliminated from further consideration due to a combination of environmental, aesthetic, safety, and engineering considerations. Detailed numerical modeling will be performed to evaluate the remaining alternatives. The GENESIS numerical model offers a valuable tool for evaluating the relative performance of alternative plans of improvement. This model will also be used to optimize many of the alternative designs prior to evaluation, particularly the plans containing combinations of structures and beach fills. The primary criteria for evaluating the relative merits of each alternative will be the reduction of erosion along the project area. This will be reflected in GENESIS modeling as the change in erosion rates relative to the existing no-action condition, defined as alternative NA-1.

Alternative NA–1. The Primary No-Action Plan (GENESIS Baseline Condition). The condition against which all GENESIS model simulations will be compared is Alternative NA-1, the No-Action plan. Under this condition the existing groin field would be left in a deteriorated state exactly as it currently exists. The project would continue to be renourished to its 'normal' 240-foot construction berm width on an as-needed basis. This alternative is shown in figure 22.

Figure 22. Plan view of Alternative NA-1 (and B-1)

Based on recent project history the average annual erosion rate along Bal Harbour has been about –55,600 cy/yr since construction of the Federal project. Since the project erodes in a non-uniform manner due to accelerated erosion at the northern end, renourishment intervals are typically shorter than would be indicated by the erosion rate alone. An analysis of survey data presented in this report shows that long-term unit erosion rates vary from a maximum of –14.9 cy/lf/yr along the northern reach of the project to a minimum of about –6.2 cy/lf/yr near the southern end of the project during the period 1990-2002. GENESIS simulations and on-site observations both confirm that fill placed along the northern reach of the project erodes rapidly during the first few years, then erosion rates decrease as the fill becomes increasingly protected by the groins and jetty. Surveys have only recently been taken on an annual basis, so yearly variations in erosion rates cannot be directly measured at this time.

Renourishment of the Bal Harbour shoreline should occur when the advance nourishment material is completely eroded; i.e. when the construction berm recedes to the design template. In many cases some erosion into the design section along the northern reach of Bal Harbour occurred long before renourishment was actually performed, since it is not considered economical to construct a renourishment project when only a small portion of the project requires fill placement. Also, as described in the preceding paragraph, the shoreline tends to stabilize as it recedes. By the time the shoreline has receded to near the design template, the annual erosion rate from this portion of the project has decreased considerably, minimizing damage to the design template. Since initial project construction in 1975, renourishments have been performed in 1990, 1998, and 2003, resulting in renourishment intervals of 15, 8, and 5 years (average 9 years).

For the purposes of this GENESIS shoreline simulation analysis, renourishment intervals will be defined as the time required for the 240-foot construction berm to erode 145 feet landward, to the seaward edge of the design template. GENESIS simulations (and actual observations) confirm that this will occur first along the northern section of shoreline between the south jetty and groin #2; the remaining areas to the south erode more slowly. Under the existing conditions of alternative NA-1, with “average” wave energy input, GENESIS simulations indicate that the shoreline will recede into the design section along the northern portion of the project in about 6 years, which corresponds reasonably well with field observations. Along the southern reaches of Bal Harbour the GENESIS-predicted renourishment interval is on the order of 15 years; this value is also corroborated in the field.

This 6-year renourishment interval will be used as a baseline for all future simulations. A primary goal of the design process is to extend the renourishment interval by more evenly distributing erosional losses along the length of the project area. Several parameters will be used to evaluate alternatives in the following GENESIS shoreline simulations. Patterns of erosion are nearly as important as the magnitude; erosional “hotspots” are to be avoided. The change in longshore transport rates and erosion rates within the project area will also be examined in each case. The alternatives which provide the best combination of shoreline response and increased renourishment interval with minimal disruption to existing longshore transport rates (minimizing downdrift effects) will be considered in the final array of plans.

Maintaining the project under the existing conditions as described in alternative NA-1 would require the placement of 333,600 cubic yards of fill every 6 years. This volume of fill would rebuild the 240-foot construction berm along the length of Bal Harbour. At an erosion rate of –55,600 cy/yr, renourishment would be initiated when the more rapidly-eroding northern section of the fill receded into the design template.

The relative lack of adequate sources of borrow material presents one of the greatest challenges to maintaining this and other segments of the Dade County BEC & HP project. All of the originally designated borrow areas for the Federal project have been depleted and/or rendered inaccessible by increasingly stringent environmental regulations. Currently the only borrow area available for use is the ebb shoal at Bakers Haulover Inlet. Availability of this borrow area depends on the condition of the ebb shoal prior to each renourishment : once used, the shoal must be allowed to refill before it can be used again to avoid damaging an important natural sediment pathway around Bakers Haulover Inlet.

At current calculated longshore transport rates, the shoal should accrete at a net rate of 60,000 cy/yr (assuming that all southbound sediment is deposited in the borrow hole). At this rate of infilling a minimum of 5.5 years would be required to refill the borrow area following a "typical" placement of 333,600 cubic yards. It is unlikely that 100 percent of the southerly-transported sediment is redeposited into the borrow area however, and the actual interval between uses of the borrow area may be considerably longer. Monitoring of the actual infilling of the borrow area will provide a more reliable estimate of the time interval required between successive uses of the ebb shoal. Annual surveys are currently being performed to determine the infilling rate from the 2003 dredging of the shoal as a borrow source for the 2003 Bal Harbour renourishment.

As an alternative to the use of the Bakers Haulover Inlet ebb shoal borrow area, an investigation is currently underway to find and develop alternate borrow sites along the southeast Florida coast, including upland sites. Several of these areas are identified and discussed in detail in the geotechnical section of this report. One of the most promising sites is a series of deepwater sand deposits along the edge of the continental shelf offshore of Dade County. Over 20 million cubic yards of beach-quality material is available from these sites in 60 to 200 feet of water. Dredging to these depths is expensive but can be accomplished with existing technology. It will be assumed that these deepwater sites will be used for the renourishment of Bal Harbour during times when the Bakers Haulover Inlet ebb shoal borrow site is not available.

The costs of using these two primary borrow sites are as follows : The estimated cost of constructing a typical renourishment of Bal Harbour as described in alternative NA-1 using the Bakers Haulover Inlet ebb shoal is $ 6,576,000. The estimated cost of constructing the same renourishment as described in alternative NA-1 using the deepwater sites offshore of Dade County is $ 6,659,000.

Alternative NA-2. Modified No-Action Plan. This alternative is similar to NA-1, but all five of the existing groins would be removed entirely. This plan will not be used as a baseline for GENESIS modeling, but is evaluated here as a potential alternative for increasing project performance. All five relic groins were removed from the GENESIS model, and the south jetty was left in its current configuration. At this point the Bal Harbour shoreline was completely unarmored. The 240-foot construction berm was used as the initial shoreline condition, and a 10-year simulation period was run to determine the length of one average renourishment cycle.

Shoreline recession increased along the length of the project under this condition. The same pattern of erosion occurred as in alternative NA-1 (and in historical observations) : erosion was the greatest at the northern end of the project, decreasing in an almost linear manner proceeding southward. With no groins in place, erosion into the design section occurred after 3 years along the northern 1,000 feet of the Bal Harbour shoreline. Renourishment intervals further south tended to be much greater – about 12 years between groins 4 and 5, near the southern limit of Bal Harbour. Due to the increased erosion at the northern end of the project, alternative NA-2 was eliminated from further consideration.

Alternative S-1. Rehabilitation of Existing Groins. The GENESIS model was set up with the five existing king pile groins in their original positions and dimensions. The layout of alternative S-1 is shown in figure 23. Since the groins have removable horizontal panels, the permeability of each structure can be adjusted as necessary. Initial simulations indicated that highly impermeable groins result in excessive erosion downdrift of the structures. This erosion pattern shifts from one side of the groin field to the other with the seasonal reversals in sediment transport direction, and results in excessive loss of berm width on the downdrift side of each structure as sediment shifts within each shoreline cell.

Groin permeability values over 50 percent provided the best results for shoreline response. Permeabilities of about 90 percent are considered to be the maximum value possible for a continuous structure. Based on simulation of the existing groin field with permeabilities ranging from 50 to 90 percent, the optimum simulated performance was achieved with a permeability value of 65 percent. The interval between successive renourishments under these conditions was about 6.5 years. A slight restriction to bypassing downdrift of the project was noted with groin permeability set at 65 percent; this downdrift effect increased substantially with lower permeability settings. Groin permeabilities less than 50 percent resulted in significant downdrift erosion. Numerical modeling does not account for two of the adverse effects of king pile structures which were observed where the horizontal connecting panels were in place : the increase in localized erosion due to wave reflection off of the vertical structures, and the generation of dangerous rip currents along the groins. Additionally, permitting agencies generally prefer stabilizing structures to be constructed using “natural” materials, i.e. rubble structures vs monolithic concrete designs. In spite of these drawbacks, alternative S-1 has some merits and will be considered further.

All construction work on alternative S-1 (as well as each of the other alternatives) would extend seaward from the existing vegetation line. The ECL is located approximately 100 feet landward of the vegetation line. Although the ECL is normally used as the baseline for work on the Federal project, the vegetation line, not the ECL, will be used as the landward limit for construction for this project. As described in previous sections of this report, the local sponsor has constructed an extensively vegetated park facility along the upper portion of the berm, and does not wish to destroy portions of the park in the process of constructing stabilizing structures. Structures constructed in the proposed manner would still be anchored deep into the design template of the Federal BEC & HP project. Since erosion of the design template will not be permitted, flanking of the structures will not occur.

Figure 23. Plan view of Alternative S-1

Construction of alternative S-1 would require excavation around the five existing structures to an elevation of –5 feet mlw. Existing broken and damaged horizontal panels would be removed to this depth. These panels are 10 feet long by 1 foot high, and an estimated 350 damaged panels would be removed and replaced with new panels. The final top elevation of the panels would be +3.0 feet mlw, or approximately +0.5 ft mhw. Damaged king piles would also be removed and replaced. Based on visual estimates, twenty percent of the piles are damaged beyond the point of repair and require replacement. Damage consists primarily of breakage to the grooves which hold the horizontal panels. Based on a total of 102 piles within the project area, 21 piles would be extracted and replaced. Piles of rubble were placed around the seaward tips of the five groins at some time in the past; this stone may also extend along the length of each groin. Much of this stone will be removed during the excavation to –5 feet mlw. Stone below elevation –5 ft mlw may be left in place.

The total volume of excavation is 5,000 cy, assuming that construction occurs when the beach is in a fully-eroded condition. In addition, the grooves in the existing king piles will require cleaning so that the new horizontal panels may be fitted in place. The estimated cost of constructing alternative S-1 is $ 1,842,000.

Alternative S-2. Construction of New Groin Field. The GENESIS model makes no distinction between the type of material which comprises the groin (or any other structure); the structure dimensions and permeability (or wave transmission factor for breakwaters) are all that the model "sees". Therefore the only difference between alternatives S-1 and S-2 as far as GENESIS modeling is concerned, is that under the provisions of alternative S-2 the groin locations, lengths, and even orientations can be changed.

To improve performance of the groin system over that which was achieved in alternative S-1 various parameters of the groin field were varied using trial-and-error approach with the GENESIS model. Several design changes were made as shown in figure 24. First, the groin spacing in the alongshore direction was changed to an even value. As seen in figure 15, the alongshore spacing of the existing structures varies from 700 feet to 1,100 feet. As a first attempt to improve performance, a uniform spacing of 850 feet was adopted. Minor changes in shoreline responses were noted, with the beach fill generally receding in a slightly more uniform manner along the southern reaches of the project. No major reduction in erosion rates resulted from the even spacing, so at this point numerous changes to the existing groin field were simulated, including adding more groins in an even distribution along the shoreline, adding extra groins near the inlet, extra groins near the suspected nodal point, and removing and/or shortening groins at the southern end of Bal Harbour to ease the transition into Surfside. Dozens of combinations of parameter adjustments were simulated in an effort to improve project performance. A detailed discussion of the results of every model run would be excessive; results of observed trends and the most promising alternatives are discussed below.

Several trends emerged during this series of runs. First, the existing five-groin system with an 850-foot spacing provides the best overall shoreline response. Larger numbers of structures, even with high assigned permeability values, will slow the southward littoral transport excessively, resulting in downdrift erosion into Surfside. On the other hand, reducing the number of groins to less than five structures results in an unacceptably uneven shoreline response, with 'updrift' areas advancing or remaining stable while 'downdrift' areas recede excessively. The five-groin system seems to provide the best overall tradeoff between slowing erosional losses and providing a relatively even recession of the project shoreline.

Various groin lengths were simulated, with lengths varying from half to nearly double the length of the existing structures. As stated previously the initial or baseline condition for all GENESIS simulations was the post-nourishment configuration, which consists of a 240-foot wide berm, with a 1v : 11h front slope. The seaward ends of each of the existing king pile groins are at approximately the same position as the post-nourishment mean sea level (msl) line, which is about 310 feet seaward of the ECL. Therefore, groins longer than the existing king pile structures would protrude from the post-nourishment beach fill, while shorter structures would be embedded within the fill. A sensitivity analysis performed on groin length showed that structures which protrude far from the fill (250 feet seaward of msl or greater) create massive downdrift erosion, with the effect tapering off as groin length approaches the berm width. Assigning a high permeability value reduces the extent of downdrift erosion somewhat for the longer structures.

Figure 24. Plan view of Alternative S-2

One difficulty that arises is that as the beach fill recedes between renourishment cycles, the effective length of any fixed groin will become longer. It was noted that over the 10-year simulation period, most groin designs have a greater effect in the last years of model simulation than in the first years, since the effective length of the structures is increased by berm recession. Due to this effect, groins with lengths approximating the post-nourishment berm width performed the best throughout the renourishment cycle even though they had relatively less effect in the first few years following renourishment. Longer structures tended to create excessive littoral blockage, especially during the last years of the cycle when berm widths had receded significantly. Groin lengths shorter than the initial berm width were ineffective for the first few years, until shoreline recession exposed the structures.

One case in which shorter groin lengths provided positive results was in the use of a tapered groin system. Groins of decreasing length (proceeding southward) were simulated, in an attempt to minimize downdrift impacts to the Surfside shoreline. Based on guidance from Corps of Engineers' Coastal Engineering Technical Notes (CETN III-12), a plan-view 6-degree taper in the seaward position of groins at the end of a groin field can be effective in reducing erosion downdrift of the structures. Applying this criteria to the layout of the structures in alternative S-2, excellent results were achieved with minimal adverse downdrift effects. In the optimal configuration, the northern three groins were the full width of the fill, with the southern two groins tapered 6 degrees as recommended. In this configuration, the length of the southernmost (shortest) groin was approximately one-half the width of the post-nourishment berm.

A sensitivity analysis was also performed on groin permeability. As expected, excessively permeable structures (85-100% permeable) had little effect on erosion, similar to the highly porous existing king pile structures. On the other hand, highly impermeable structures (0-40 percent permeable) tended to trap excessive amounts of material, causing downdrift erosion. The target zone for permeability was therefore 40-85 percent. The higher values of this range had little impact in substantially reducing erosion rates, while the lower values in this range tended to produce larger downdrift/updrift fluctuations in shoreline position near some groins. The median permeability values of 60 - 70 percent provided a good tradeoff between reducing erosion and allowing a uniform shoreline response.

The final plan developed under alternative S-2 consists of five rubble-mound groins spaced evenly at 850-foot intervals along the Bal Harbour shoreline. The northern three structures will be the same length as the renourished berm width, and the southern two structures will be tapered 6 degrees in length. Individual groin lengths from the ECL will therefore vary from 310 feet at the north end of Bal Harbour to 180 feet at the south end of Bal Harbour. All five groins will have a permeability of about 65 percent, which can be achieved by constructing the low-crested groins entirely of armor stone, with no core. The GENESIS-predicted renourishment interval for alternative S-2 is 6.8 years. This alternative provides an improved shoreline response and will be examined in greater detail.

The design of the rubble-mound structures is described as follows. The five existing king pile groins would be replaced by five rubble-mound groins. Rubble groins would be constructed along the existing alignments of groins 1, 2, 3, and 5. Groin 4 would be rebuilt 100 feet south of its present location. Little information is available on the design of the existing piles, particularly the depth of embedment. Most of the king piles are completely buried in the beach fill and even the overall condition of these piles is not known. Therefore either of two methods of construction are determined to be acceptable, depending on the practicality of removing the piles. The piles should be removed intact if possible but if the depth of embedment is such that pile extraction is not reasonably practical, the piles at groins 1, 2, 3, and 5 may be cut off at elevation +1.0 ft, mlw. In the latter case, foundation materials and armor stone would be placed over and around the cut-off king pile structure. The king piles at groin #4 would be removed entirely or cut at an elevation of –5 ft mlw, and the new structure constructed 100 feet to the south to provide a more uniform groin spacing along the project area. Due to the shorter length of the rebuilt groin 5, each of the estimated 12 existing piles seaward of the rebuilt end of groin 5 would be removed entirely, or cut at a depth of at least –5 ft mlw.

The intent of these different cut-off depths is as follows : In areas where the existing piles fall outside of the proposed footprint of the new structures, a deeper cut-off depth of –5 ft mlw is specified, to ensure that scouring does not expose the piles and endanger bathers in the area. In areas where existing piles fall within the footprint of construction of the rebuilt groins, a shallower cut-off depth of +1.0 ft mlw is allowed to facilitate construction. Public safety would not be compromised because the cut-off piles would be embedded within the new structures.

Following pile removal as described above, each of the structures would be excavated to a depth of –3 feet mlw, coinciding with the maximum expected depth of scour observed from previous monitoring surveys. Woven geotextile fabric would be placed beneath the footprint of each structure in the excavated areas, then marine mattresses would be placed on top of the geotextile cloth to form the structure's foundation. Finally, armor stones would be placed directly on these mattresses to construct the permeable rubble groins. Based on maximum expected breaking wave size at the structures a median armor stone size of 1.2 tons is required. The crest width would be 7.5 feet with side slopes of 1v : 1.5h. The armor layer would extend from a top elevation of +4 feet mlw down to the top of the foundation mats at elevation –2 ft mlw.

The total volume of excavation required is 3,000 cubic yards. A total of 3,450 square yards of geotextile fabric is required, and 3,250 square yards (2,000 tons) of bedding mattresses. The total volume of 1.2-ton, 165 pcf armor stone required for construction of the five groins is 7,140 tons. The estimated cost of constructing alternative S-2 is $ 2,265,000.

<u>Alternative S-3. Construction of T- (or "Tuned-") Groin Field</u>. This option would be similar to option S-2, but with the addition of T-heads on some or all of the groins as shown in figure 25. The T-segments on the end of groins can increase performance of the groin field by reducing bypassing, by more effectively holding material between the structures, and by reducing rip currents and subsequent losses of fill into deep water. The T-head groin design has been used in similar locations in Florida to contain downdrift losses near inlets and is well-suited to this project area, particularly in regards to holding sediment along the erosive northern end of the project.

The newest version of GENESIS-T was used to simulate the addition of T-head structures. Previous versions of GENESIS do not allow the simulation of T-head groins, or any combination breakwater/groin structure. The same series of model simulations from alternative S-2 were performed using various combinations of T-head configurations. The guidelines developed during simulations for alternative S-2 relating to structure lengths, permeabilities, position, etc were applied to the T-head structures.

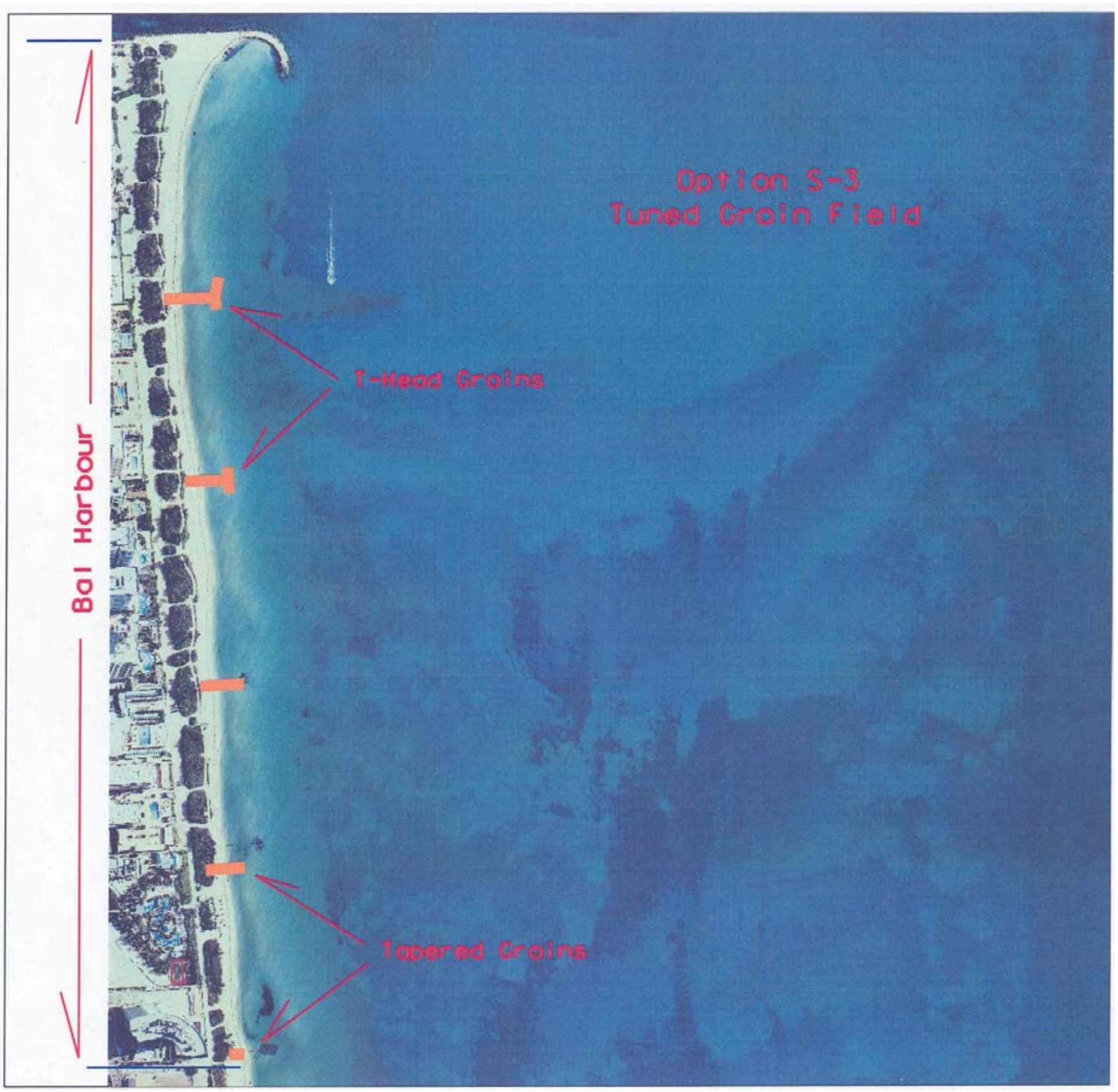


Figure 25. Plan view of Alternative S-3

In general, it was noted that the T-head structures held beach fill between the groins more effectively than the non T-head structures from alternative S-2. In some cases this effect was detrimental, such as along the south end of the fill, where the retention of fill resulted in increased downdrift erosion. The main advantage of the T-heads was apparent at the north end of the project. As discussed previously, this region along the northern 2,000 feet of Bal Harbour is the most rapidly eroding portion of the project. Material is transported out of this area in both directions, with a large percentage of losses due to sediment transport northward around the jetty and into the inlet. T-head groins would be used along this area to retain fill on the beach more effectively than the non T-head groins. The T-head structures generally maintain the shoreline position at a point further seaward than non T-head groins along the northern Bal Harbour shoreline.

A similar iterative procedure as was used to develop alternative S-2 was used to optimize the configuration of T-groins for alternative S-3. T-heads were simulated on various combinations of groins, and in addition a wide variety of groin lengths, permeabilities, and spacings were simulated. The groin field layout with the most favorable shoreline response consisted of a design similar to alternative S-2, but with T-sections on the northern 2 groins.

The final design for alternative S-3 consists of a five-groin system, with an average groin spacing of 850 feet, and groin permeabilities of 65 percent. The northern three groins extend seaward to the post-nourishment msl line, and the southern two groins are tapered 6 degrees to reduce downdrift effects. The T-head on the northern groin (groin #1) will extend 50 feet to the north and 25 feet to the south from the seaward tip of the structure. The northern T will be angled 10 degrees toward seaward, to face directly into the average incident wave direction (from the STWAVE analysis). The southern segment of the T on groin #1 will be shore-parallel. The T-segments on groin #2 will each be 25 feet long, and will both be oriented in a shore-parallel direction. Groins #3, #4, and #5 will follow the same design as in alternative S-2.

The predicted renourishment interval for alternative S-3 is 8.5 years along the area north of the nodal point, and over 15 years along the shoreline south of the nodal point. The increased renourishment interval (compared to alternatives S-1 and S-2) is due to the T-heads' ability to better hold material within the shoreline cells between the south jetty and groin 1, and between groin 1 and groin 2. This alternative could represent a substantial cost savings, as the southern portion of the Bal Harbour could be renourished during every second renourishment. Due to the more favorable performance of the T-head structures versus the straight rubble-mound structures, alternative S-3 is recommended for further investigation.

The design of alternative S-3 is essentially the same as that described for alternative S-2 except for the addition of T-heads on groins 1 and 2. As in alternative S-2, groins 1, 2, 3, and 5 would be built over the existing king pile structures, while groin 4 would be rebuilt 100 feet south of it's present location. The stone sizes and cross-sections would be identical to those proposed in alternative S-2. As in alternative S-2, all existing king piles seaward of the vegetation line would be removed, or cut at the grades specified. Due to the shorter length of the rebuilt groin 5, each of the estimated 12 existing piles seaward of the rebuilt end of groin 5 would be removed entirely, or cut at a depth of at least –5 ft mlw.

The total quantities of stone required to construct alternative S-3 are as follows. The quantity of 1.2-ton armor stone is 6,252 tons, quantity of foundation mattresses is 1,759 tons (= 2,870 sq yds.), and quantity of geotextile fabric is 3,032 sq yds. The total volume of excavation is 4,500 cy. The estimated cost of constructing alternative S-3 is $ 2,768,000.

Alternative S-4. Construction of Offshore Breakwaters. Several alternative breakwater alignments were simulated in a series of GENESIS runs to determine the optimum breakwater layout. Initial runs were modeled loosely on the successful Sunny Isles structure (located 4 miles north of the study area), and consisted of a series of 400-foot breakwater segments separated by 200-foot gaps, extending along the length of Bal Harbour. It was quickly determined that breakwaters south of the nodal point tended to retain excessive amounts of sediment and created downdrift erosion, so subsequent runs eliminated or reduced the number and/or effectiveness of breakwater segments in this region. The breakwater segments between the nodal point and the south jetty seemed to provide the greatest benefits in terms of shoreline protection. No breakwater construction was proposed in the region where the ebb shoal reconnects with the shoreline (between groins 1 and 3), in order to avoid interrupting the onshore transport of sediment from the shoal.

Various combinations of breakwater segment lengths, gap lengths, crest elevations, and distances offshore were simulated. The optimum design consisted of a series of three breakwater segments, beginning 200 feet south of the south jetty and extending southward to near the nodal point. Each segment would be 200 feet in length, separated by a 200-foot gap. The structures would be located approximately 350 feet seaward of the post-nourishment mhw line, or 660 feet seaward of the ECL. A plan view layout of the proposed design is shown in figure 26. The design crest elevation is –3 feet, mlw, the same elevation as the Sunny Isles breakwater. This crest elevation results in a structure which would be largely invisible from the shore, an important consideration for the area’s aesthetics. This crest elevation also makes the structure somewhat of a navigation hazard, but lower crest elevations would make the breakwater ineffective. Navigational warning buoys were required by the U.S. Coast Guard at the Sunny Isles breakwater, and would also be required at any submerged offshore structure at Bal Harbour due to the large number of boaters in the proximity of the nearby inlet.

GENESIS simulations of the proposed design indicate a reasonable shoreline response, with relatively even shoreline recession along most of the project area. Three stable salients form in the lee of the structures, and these salients tend to stabilize the shoreline between the jetty and the nodal point. Shoreline response south of the nodal point is uniform. Construction of this alternative results in a simulated renourishment interval of 7.25 years along the northern reach of Bal Harbour, and up to 15 years along the southern reaches. As in alternative S-3, a substantial cost savings may result, as the southern portion of the Bal Harbour could be renourished during every second renourishment.

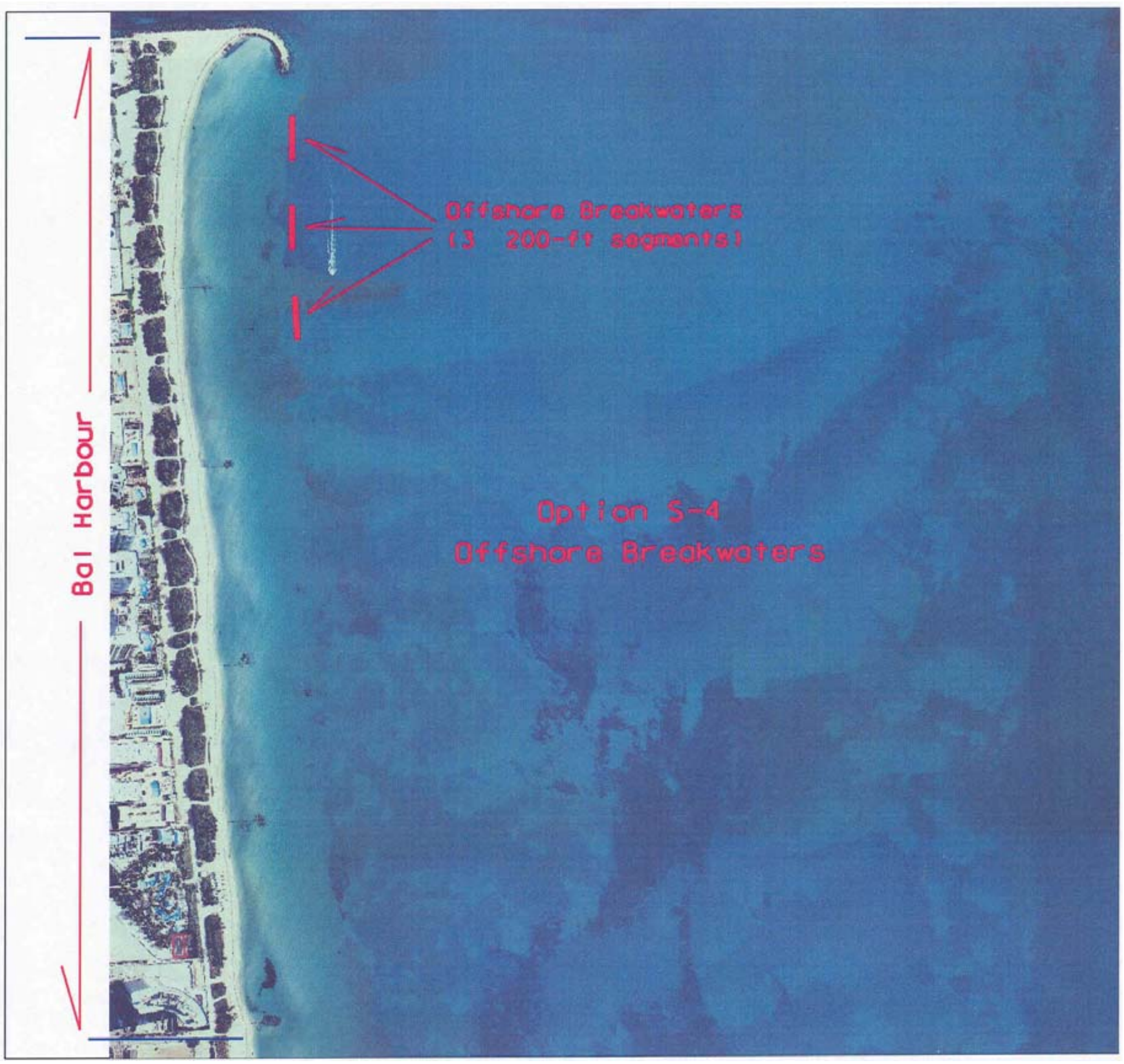


Figure 26. Plan view of Alternative S-4

The breakwater would be constructed of rubble, with the foundation layer designed to protect against scour. The median armor stone size would be 5 tons, and the crest width would be 12 feet at elevation –3 feet mlw. Side slopes would be 1v : 1.5h. An alternative to the rubble-mound design would be any of a number of innovative technologies which use materials such as artificial reef modules as the main body of the breakwater structure. As previously discussed, the GENESIS numerical model does not distinguish between the various types of construction materials; the degree of wave transmission over/through the structure is all the model "sees". Specifics of the design of innovative breakwater technologies will be discussed under alternative I-2.

The simulated shoreline response of the optimum breakwater configuration is better, in terms of renourishment interval, than the performance of the groin field in alternative S-2. In spite of the favorable shoreline simulations, construction of breakwaters in this location may be inadvisable. In addition to the navigational hazard created by the structures (discussed above), other adverse impacts could exist. The proposed location will place the structures (particularly the northern breakwater segment) within the area of influence of tidal currents flowing through Bakers Haulover Inlet. The shore-parallel orientation of the breakwaters may channel tidal flow along the beach, resulting in increased scouring of the shoreline (which GENESIS is not able to predict) and increased danger to bathers in this heavily-used swimming area. In addition, tidal scouring in this location would likely result in erosion of the seafloor in the vicinity of the breakwaters, possibly undermining portions of the structure.

Another consideration in the construction of offshore breakwaters is the presence of hardbottom or other environmentally sensitive areas in the nearshore zone. Present data indicates that no coral reef areas exist within the limits of construction of the breakwaters, but a detailed dive survey should be conducted to determine if any other environmental resources are present in the area. In spite of the many practical difficulties of constructing breakwaters in this location, the positive effects of this breakwater design on shoreline response justify further investigation of this alternative.

Construction of the three 200-foot breakwater segments described above would require the placement of 11,856 tons of 165 pcf, 5-ton armor stone, 2,043 tons of marine foundation mattresses, and 3,450 sq yds of geotextile fabric. A total of 2,000 cubic yards of excavation would be required. The estimated cost of constructing alternative S-4 is $ 3,637,000.

<u>Alternative S-5. Combination of Rubble Groins and Breakwaters</u>. Any number and configuration of rubble groins and breakwaters could be combined to create an effective solution to the erosion at Bal Harbour. As a general plan, breakwaters would be used near the north end of the project to shield that region from the wave energy focusing predicted by the wave refraction analysis in this report. Groins would be used south of the nodal point to reduce the southward-directed losses of fill material. Various combinations of structures were devised and tested using the GENESIS numerical shoreline change model.

As discussed in alternative S-4, optimum performance of the segmented breakwater alternative was achieved by using a three-segmented breakwater, located entirely north of the nodal point. This design achieved maximum stability of the northern segment of Bal Harbour, with a renourishment interval of about 7.25 years, and a renourishment interval of about 15 years for the southern segment of Bal Harbour. Since the renourishment interval of the southern segment is nearly double that of the northern segment, a preliminary conclusion is that no additional structures are needed along the southern portion of the shoreline.

However, simulations of several alternative layouts of groins south of the nodal point were performed nonetheless, in order to determine if a further increase in renourishment interval would offset the added cost of constructing such structures. Lessons relating to groin lengths, spacings, and permeabilities were applied to the region south of the nodal point, and the optimum layout that resulted from GENESIS simulations was similar to the southern two groins in alternative S-3. The northern groin would extend seaward to the post-nourishment mhw line. The southern groin would be tapered 6 degrees landward, to reduce downdrift effects. The resulting renourishment interval with the three breakwater segments and two groins was still 7.25 years along the northern shoreline, but increased to about 16 years along the southern shoreline.

This minimal extension of the renourishment interval along the southern Bal Harbour shoreline does not provide a significant cost savings, since the southern shoreline could already be renourished during every other renourishment contract under several of the proposed alternatives. In effect, the only benefit which would accrue would be that the volume of renourishment along the southern shoreline would be slightly less under alternative S-5 than alternative S-4. Since this proposal represents a significant increase in construction cost for a minimal reduction in renourishment volume, this alternative is not economical and will not be considered further.

<u>Alternative S-6. Extension of the Bakers Haulover Inlet South Jetty.</u> The existing south jetty at Bakers Haulover Inlet would be lengthened by up to several hundred feet. Extending the jetty would allow a larger volume of material to be impounded at the northern end of Bal Harbour, in the area where erosion is usually the most severe. The curve at the seaward end of the existing jetty has proven useful at reducing the formation of large vortices (especially on ebb tide) which can adversely affect the shoreline, and at reducing scouring near the seaward tip of the structure due to tidal currents. For these reasons the curved seaward end of the jetty would be reconstructed at the end of the new structure.

The GENESIS and STWAVE models were used to test various increments of jetty lengthening. Increments of 100 feet were added to the existing jetty length for each GENESIS simulation, and shoreline response, longshore transport values, and volumetric changes were evaluated. The existing condition of the groin system was used in all GENESIS simulations (alternative NA-1). Since net sediment transport is from north to south, it is reasonable to assume that the north jetty would be extended seaward by a similar distance, in the interest of preventing excessive channel shoaling and maintaining the navigability of the inlet. The cost of any north jetty extension will be excluded from this analysis however.

Model results showed some improvement in shoreline response, mainly along the area of most severe erosion, north of groin #1. GENESIS simulations were performed for south jetty extensions of 100, 200, 300, 400, and 500 feet. Jetty extensions of 100 and 200 feet provided only minimal improvements to shoreline response, 300 feet provided a very stable shoreline response, while the longer extensions of 400 and 500 feet offered about the same shoreline response as the 300-foot extension, but at a greater cost due to the longer structures.

A 300-foot extension of the south jetty provided the optimum shoreline response : erosion between the south jetty and existing groin field was reduced to the point that the renourishment interval increased from 6 years to about 7 years. The shoreline positions along northern Bal Harbour fluctuated much less with the longer jetty extensions in place, compared to the shoreline fluctuations under existing conditions. The mechanics for this process are straightforward : a larger filet builds up against the south side of the south jetty during the summer months, when longshore transport is directed northward. During the winter months material from this filet is re-distributed along the northern section of the beach, offsetting the erosion normally observed in this area. The following summer this process repeats itself. A plan view of alternative S-6 is shown in figure 27.

Figure 27. Plan view of Alternative S-6

Some adverse impacts could result from the jetty lengthening however. Each of the jetty extensions will displace the jetting effects of tidal flow further seaward. This will in turn increase the degree of wave energy directed toward the inlet (especially during ebb flow) as was demonstrated in the preceding wave refraction analysis. The increased wave energy along the shorelines in the vicinity of the inlet will very likely result in increased beach erosion, which is not considered in the preceding GENESIS modeling. It is therefore considered very likely that the preceding GENESIS analysis will under-predict shoreline erosion in the vicinity of the inlet. It is also highly likely that the ebb shoal will be displaced seaward by a distance similar to the length of the extension, based on a parallel jetty alignment. This could result in a further decrease in natural bypassing that is difficult to quantify. In spite of these potential negative effects, alternative S-6 shows positive impacts on the northern Bal Harbour shoreline and will be further investigated.

The existing south jetty cross section would be used as a design template for extending the jetty. The existing crest width is about 20 feet, at an elevation of about 8 feet mlw. This cross-section is reasonable for the proposed design, considering the large armor stone size required due to the structure's vulnerability to large depth-limited waves. Depths along the alignment of the proposed jetty extension range from about –8 to –18 feet mlw. The proposed structure is sheltered somewhat by the shallower depths along the ebb shoal; minimum controlling depths along the shoal are about 12 feet. Adding the effects of a 2.5-foot tide and a 4.5-foot 10-year storm surge yields a maximum controlling depth for depth-limited waves of (12' + 2.5' + 4.5') = 19 feet. Applying the relation $H_b = 0.78d$ yields a maximum depth-limited design wave height of 15 feet. The corresponding median armor stone size according to Hudson's equation is 22 tons.

In order to prevent undermining, the foundation of the structure would be constructed to the depth of maximum expected scour, -18 feet mlw. The foundation would consist of marine mattresses with underlying geotextile fabric. Side slopes of 1v : 1.5h would be used throughout the structure. An intermediate stone layer would be required due to the large size of the armor stone. Intermediate stone size would be 2 tons, and would be placed between the foundation mattress and the armor stone layer.

The quantities of materials required to construct alternative S-6 are as follows : A total of 3,100 cubic yards of excavation would be required, followed by the placement of 1,970 tons of marine mattresses to form the structure's foundation. Total volume of intermediate stone would be 13,130 tons; the volume of armor stone would be 16,300 tons. The 150-foot curved section of the seaward end of the existing jetty could be left in place as an additional measure for stabilizing the north end of the project, and for use as a fishing platform. The total estimated cost of constructing alternative S-6 is $ 8,661,000.

Alternative S-7. Construction of Sand Bypassing Facility. A fixed bypassing facility would be constructed, similar to the existing facility at Lake Worth Inlet in Palm Beach County. The plant would pump material from an impoundment area on the north side of Bakers Haulover Inlet via pipeline to a pumpout area along Bal Harbour, as shown in figure 28. Such a facility would increase the rate of bypassing around the inlet and would decrease the erosion along the Bal Harbour shoreline due to the disruption of littoral transport caused by the inlet. This plan could include extension of the north jetty, to increase impoundment capacity. Initially investigated in the Bakers Haulover Inlet Management Plan (IMP), this plan was not recommended for further investigation due to high cost and the adverse impacts on recreation and aesthetics of the area. The IMP was published in 1995 however, and the concept of mechanical sand bypassing will be re-evaluated in this report.

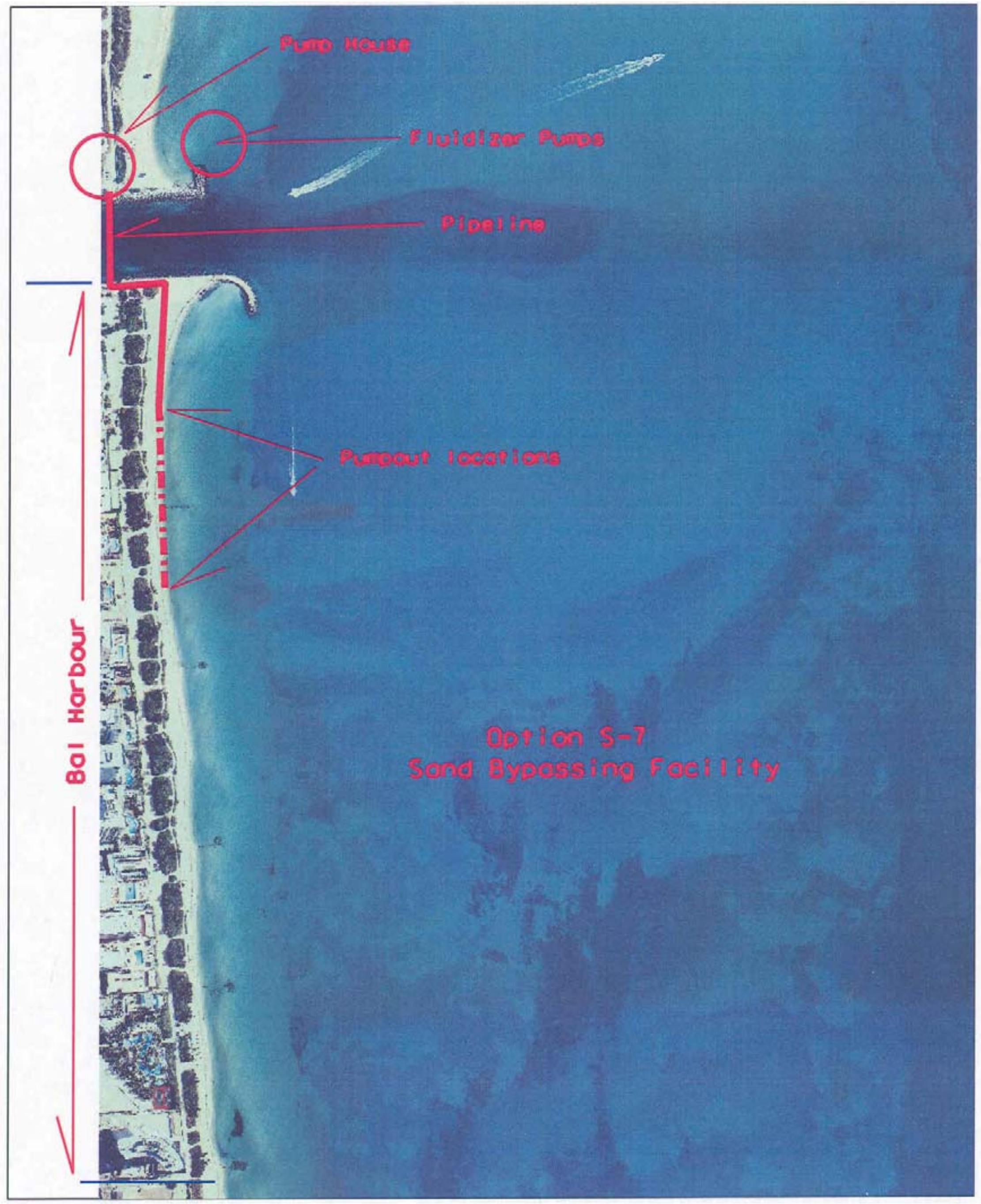


Figure 28. Plan view of Alternative S-7

The design for this plan will be very similar to the proposal submitted in the 1995 report, which was in turn based on existing bypassing facilities at Lake Worth Inlet and South Lake Worth Inlet in Palm Beach County. A fixed plant would be constructed near the base of the north jetty, and two fluidizer/jet pumps would be installed on the existing spur of the north jetty. The jet pumps would be positioned along the ocean floor in the vicinity of the jetty spur; one pump in the intertidal zone and the other pump along the outer surf zone. Each jet pump would excavate a sediment/slurry mixture which would be transferred via booster pumps through a pipeline to the south side of the inlet. Discharge points in the pipeline would be positioned on either side of the nodal point, approximately 500 and 1250 feet south of the inlet. Approximately 3,000 linear feet of pipeline would be required to transport bypassed sediment from the north jetty to the two discharge points along the Bal Harbour shoreline. As shown in the sediment budget developed in this report, an average volume of –55,600 is eroded from the Bal Harbour shoreline; this will be the target bypassing rate for the sand bypassing plant.

This target bypassing rate was input into the GENESIS shoreline change model and 10-year simulations were run using the existing site conditions. These simulations showed that the project shoreline can be maintained almost indefinitely by bypassing material to northern Bal Harbour at a rate of 55,600 cy/yr. The optimum shoreline response was achieved with 60 percent of the discharge to the north of the nodal point; 40 percent to the south. There is some concern that a large percentage of the bypassed material may be rapidly re-deposited into the inlet. A lengthening of the south jetty could reduce the volume of transport into the inlet as was shown in alternative S-6. Numerical modeling indicates that a 100-foot extension of the south jetty will greatly reduce the transport of bypassed sediment into the inlet, allowing the annual bypassed volume to be reduced to approximately 42,500 cy/yr, with the same shoreline response noted above.

Several practical problems are associated with this alternative. Bypassing plants require a physical location, which in this case would be within Haulover Park, immediately north of the inlet. The construction of the plant would require the loss of a portion of the park for recreational use. Rights-of-way are also required for the pipeline easement, which would extend across the inlet and along the upland region of the Bal Harbour shoreline. Construction costs and maintenance costs are high for bypassing plants due to the corrosive effects of salt water and the abrasiveness of the bypass slurry. Other problems to be overcome are the noise generated by the bypassing pumps and by the equipment maneuvering the jet pumps, and the degradation of the area's aesthetics caused by the fixed plant and by the discharge pipeline and associated machinery along the Bal Harbour shoreline.

In spite of these drawbacks, sand bypassing is recommended for further investigation due to the favorable shoreline response, and due to its compatibility with regional sediment management principles. The estimated cost of constructing the sand bypassing facility is $ 4,705,000, plus annual maintenance costs of $ 279,200.

<u>Alternative S-8. Close Bakers Haulover Inlet</u>. The two existing ocean jetties would be dismantled, and stone from the jetties and sediment from the ebb shoal would be used to fill in the inlet throat. Natural littoral transport along the shoreline would be restored and localized erosion on both sides of the inlet would be reduced or eliminated. In addition, the ebb shoal would slowly migrate toward the shore, eventually depositing over a million cubic yards of beach-quality material into the littoral system.

This alternative was originally investigated in the Bakers Haulover Inlet Management Plan (reference 2c). Although this alternative would be very effective at restoring the uninterrupted flow of sediment along the shoreline, it was found to be unacceptable for numerous environmental and political reasons. The original purpose of constructing the inlet was to improve water circulation in northern Biscayne Bay and the other connecting interior bays and waterways. Closing the inlet would greatly decrease tidal flushing throughout all of these interior waterways, and would cause so many adverse environmental impacts to the seagrass, mangrove, and other saltwater habitats inside the bay, that this alternative can not be considered as implementable.

Another primary purpose of the inlet was to allow navigation between the Intracoastal Waterway and the Atlantic Ocean. Bakers Haulover Inlet serves as the only ocean access between Government Cut (Miami Harbor) and Port Everglades (Ft Lauderdale), and is very heavily used by all types of commercial and recreational watercraft. This valuable ocean access would be lost and a large number of boaters adversely affected if the inlet were closed. For environmental, navigational, and practical reasons the Inlet Management Plan recommended that this alternative not be considered further, and that recommendation is supported in this report as well.

Beach Fill Alternatives. The GENESIS shoreline change model was used to evaluate the following beach fill alternatives. The existing site conditions (five relic king pile groins, Bakers Haulover Inlet south jetty) were used as a basis for all model runs, so that only the effects of the different beach fill configurations would be apparent.

<u>Alternative B-1. Construction of "Historic" Beach Fill</u>. This option would result in the reconstruction of the original construction berm, which consists of a template 240 feet wide at elevation +9.0 feet mlw, with a front slope of 1v:11h. This beach fill template offers the advantage of being fully permitted, and performance of beach fill placed according to this design is well documented in this (and other) reports. This beach fill configuration was used as the baseline condition for GENESIS simulation of the existing shoreline (alternative NA-1 – see figure 22). Model simulations showed that the average renourishment interval under existing conditions is about 6 years.

Based on the historic loss rate of –55,600 cy/yr, each project renourishment would require the placement of 333,600 cubic yards of fill. This volume of fill would rebuild the 240-foot construction berm along the length of Bal Harbour. As discussed under alternative NA-1, borrow sources are currently very limited along the Dade County coast. The primary borrow source for this material would be the ebb shoal at Bakers Haulover Inlet, when available. The secondary borrow source is the series of deepwater sites along the edge of the continental shelf. The estimated cost of constructing a project renourishment as described in alternative B-1 (same as NA-1), using the Bakers Haulover Inlet ebb shoal is $ 6,576,000. The estimated cost of constructing the same renourishment using the deepwater sites offshore of Dade County is $ 6,659,000.

Alternative B-2. Construction of Beach Fill of Altered Dimensions. This option would be similar to option B-1, except that the fill dimensions could be modified to optimize the performance of the fill. The dimensions that could be modified include the front slope of the berm, berm elevation, and berm width. The equilibrium front slope of the beach is dependent on the characteristics of the fill material, and experience has shown that most fill will equilibrate at about a 1v : 11h front slope along the Dade County shoreline. The berm elevation was formulated based largely upon existing upland elevations and storm wave runup. Both the front slope and the berm width of the beach are considered to be fixed, leaving berm width as the only variable dimension.

Practical limits to the range of allowable berm widths are imposed by the physical processes along the project area. Berm widths narrower than the 240-foot “historic” berm width would result in shorter renourishment intervals and would be counterproductive. Significantly wider berm widths displace the seaward edge of the fill into deeper water where the material is exposed to higher wave energy and greater tidal current scouring. In addition, considerable permitting difficulties may be encountered when extending the footprint of the beach fill area beyond that which is currently specified in the permit. If environmentally significant hardbottom areas exist near the seaward toe of the fill, the option of widening the berm may not be possible.

As the berm width is extended further seaward, a greater volume of fill is required for each additional increment of berm width, since the toe of fill is placed in progressively deeper water. Additionally, the short length of the south jetty at Bakers Haulover Inlet limits the berm width in the vicinity of the inlet : the existing toe of fill extends to near the seaward end of the jetty under the existing condition. Extending the fill width beyond the limit of the south jetty results in accelerated erosion along the northern reach of the project, and is not recommended. For these reasons, it is advised that the historically-placed berm width of 240 feet along the length of Bal Harbour represents the maximum practical berm width and should not be changed. GENESIS modeling shows that a more effective option may be to place additional fill only along the portions of the project which experience the most rapid erosion. This option will be explored in Alternative B-3.

Alternative B-3. Construction of Feeder Beach. Additional beach fill would be placed along selected areas of the project, in order to extend the renourishment interval. The most promising design for alternative B-3 is to place additional fill in the vicinity of the nodal point, allowing wave action to spread the material naturally through the project area. This additional volume of material would act as a feeder beach to the 240-foot “historic” berm design.

Based on GENESIS modeling, a minor increase in renourishment interval is achieved by placing additional fill in the vicinity of the nodal point, in the vicinity of groin 1. Due to the discontinuity in berm width, and due to the berm placement in deeper water, erosional losses from this wider berm are greater than for the 240-foot berm, and losses increase greatly as the feeder berm is widened beyond about 150 feet. GENESIS modeling indicates that stabilizing structures are required for feeder berm widths in excess of 200 feet. A further complication is the proximity of the nodal point to the inlet. Assuming the feeder beach is centered on the nodal point (located from 600 to 1,100 feet south of the south jetty), the northern end of the fill will be placed adjacent to the jetty, where greatly accelerated losses are expected to occur due scouring from tidal currents.

The optimum design of the feeder beach was achieved using a trial and error approach with the GENESIS model. Optimum shoreline response was attained with a feeder beach berm width of about 150 feet (in addition to the 240-foot berm), which is near the maximum berm width which can be maintained without additional stabilizing structures. The feeder beach extends from 300 feet south of the inlet to 1,500 feet south of the inlet, and would require the placement of 137,000 cubic yards in addition to the volume required to reconstruct the 240-foot construction berm. The addition of this feeder beach would result in a renourishment interval of about 6.75 years. This alterative provides favorable shoreline response and will be considered further, but construction of the proposed feeder beach would be contingent on the results of a survey of nearshore environmental resources.

Construction of this alternative would require the placement of 470,600 cubic yards of fill. The cost of constructing this alternative using fill from the Bakers Haulover Inlet ebb shoal borrow site is $ 9,277,000. The cost of constructing the fill using the deepwater borrow site is $ 9,393,000.

Alternative B-4. Construction of Nearshore Berm. Beach fill would be placed in the nearshore region, along selected reach(es) of the project area. This fill configuration would serve two purposes : to provide a source of sediment for onshore migration, and to act as a low-crested breakwater, reducing wave heights along the beach.

The nearshore placement would supplement the 240-foot construction berm, not replace it. This is due to the need to establish the design berm to fulfill the purpose of the Federal BEC&HP project - to provide storm damage protection. In addition, placing an adequate volume of nearshore fill to achieve a renourishment interval in excess of the current 6-year interval would require the coverage of a very large area of nearshore bottom, a proposition which would be environmentally unacceptable. The proposed design for nearshore fill placement would consist of creating a submerged berm extending along the same reach of shoreline as the feeder beach developed in Alternative B-3. GENESIS shoreline modeling was used as a basis for predicting sediment movement for that alternative, and the transport of sediment placed in a nearshore berm along the same reach of shoreline should follow a similar pattern.

Fill would be placed along the seaward edge of the nearshore sandbar following construction of the 240-foot berm. The crest elevation of the sandbar averages -3 feet mlw, and the nearshore berm would match that elevation and extend the width of the sandbar 310 feet seaward, with the seaward toe of the berm roughly following the 16-foot depth contour. The volume placed in the nearshore berm would be 137,000 cubic yards, approximating the volume of the feeder beach in Alternative B-3. The nearshore berm would extend along the same limits as the feeder beach : 300 to 1,500 feet south of the south jetty.

The entire volume of the nearshore berm lies within the depth of active sediment transport, and under the best scenario would mimic the performance of the feeder beach described in alternative B-3. Under the "worst-case" scenario, performance of the berm would mimic that of existing conditions. There is uncertainty due to modeling limitations of nearshore submerged berms, but it can be assumed that the renourishment interval for alternative B-4 should be within the range bounded by the existing conditions (6 years) and by the feeder beach proposed in alternative B-3 (6.75 years). The use of a nearshore berm shows some potential as an alternative to conventional beach fill placement and will be investigated further.

The construction of the 240-foot construction berm supplemented by the nearshore berm would require the placement of 470,600 cubic yards of fill. The cost of obtaining the material from the Bakers Haulover Inlet ebb shoal borrow site is $ 9,277,000. The cost of obtaining the material from the deepwater borrow site is $ 9,393,000. Note that the volumes and costs for constructing the nearshore berm are identical to those for construction of the feeder beach.

<u>Alternative I-2. Breakwater with Artificial Reef Modules</u>. An offshore breakwater would be constructed similar to the design proposed in alternative S-4, but instead of armor stone the body of the structure would be constructed using concrete artificial reef modules. The structure would have a wider crest to offset the higher porosity of most types of reef modules (as compared to armor stone). In addition to providing shore protection via reduced wave energy along the shoreline, the structure would also provide recreational and environmental benefits as a nearshore artificial reef. The selected plan for the Miami Beach site in the Section 227 Innovative Technologies demonstration program consisted of a nearshore breakwater constructed of Reefball[R] artificial reef modules. The design of alternative I-2 would be similar to the design of that structure.

The low-crested and highly porous nature of this structure makes numerical modeling of alternative I-2 very difficult, and results of modeling may have a high degree of uncertainty. The local sponsor has indicated some safety concerns with placing artificial reef modules in a location close to the strong tidal currents of Bakers Haulover Inlet. Additional safety concerns are raised regarding the placement of shallow, submerged reef modules near Bakers Haulover Inlet, due to potential boat collisions with the reef.

A version of the structure proposed in alternative I-2 is scheduled for construction in 2005 under the authority of the Section 227 "Innovative Technologies" program along northern Miami Beach, three miles south of Bal Harbour. It is recommended that the monitoring of that structure be used as a basis for constructing a similar design in other comparable locations. It is recommended that this alternative (and other "innovative" designs currently under testing in the Section 227 program) not be constructed until proven effective in the monitoring portion of that program.

Summary of Alternative Plans of Improvement. Eighteen alternative plans of improvement were presented in the previous section of this report, including two "no-action" plans. Three of these alternatives were eliminated initially due to environmental and/or public safety concerns. These plans were :

Plan	Description	Reason for elimination
B-5	Perched Beach	Environmental, public safety
I-1	Porous (fabric) Groins	Environmental, uncertain performance
I-3	Beach Mats	Environmental, aesthetics

The remaining plans were found to conform better to environmental, safety, and aesthetic standards. Numerical simulations were performed on many of these alternatives. Of the remaining fifteen alternative plans of improvement, another five were eliminated based on performance criteria, including adverse impacts on other nearby Federal and non-Federal projects. These five alternatives are summarized below :

Plan	Description	Reason for elimination
NA-2	No Action, Remove Kingpiles	Poor performance
S-5	Combination Groins + Breakwaters	Poor performance vs cost
S-8	Close Haulover Inlet	Incompatible w/Fed nav project
B-2	Beach Fill w/Altered Dimensions	Not cost effective
I-2	Reef Module Breakwater	Safety,design undergoing testing

The ten remaining plans (including one "no-action" plan) were developed in greater detail, including formulating detailed project designs for each alternative. Quantities of materials were calculated for construction of each alternative and detailed cost estimates were obtained. Tables 19 through 21 provide a summary of each of the ten remaining alternatives, the estimated construction cost of each plan, predicted renourishment intervals, operation and maintenance costs, and average annual equivalent costs over the remainder of the project life. These values will be used as a basis for selection of the recommended plan in the following economic analysis.

Economic Analysis.
The primary goal of this study is to develop a plan of improvement for the Bal Harbour study area which will reduce the high rates of beach erosion currently observed along that shoreline. Of the eighteen alternative plans of improvement originally proposed, nine alternatives (plus the no-action plan) have been found to be acceptable in terms of expected project performance, acceptability of environmental impacts, and compatibility with basic public safety and aesthetic requirements. The eight remaining proposed plans of improvement will now be evaluated economically by comparing the "with-improvement" costs to the "without-improvement" costs.

Description of Methodology. Average annual equivalent costs will be used as a basis for comparison in this economic analysis. These costs will be developed for each alternative plan based on an amortization of the estimated construction costs, plus an amortization of future maintenance costs. Maintenance costs for most alternatives will consist of periodic renourishment, with the renourishment interval predicted by the GENESIS numerical shoreline simulation model. Structural maintenance costs are included where applicable. The estimated construction costs of each alternative are summarized in table 19. Note that each beach fill alternative includes cost estimates for each of the two primary borrow sites : Bakers Haulover Inlet ebb shoal and the deepwater sites off Dade County.

Table 19
Summary of Costs – Final Array of Alternative Plans of Improvement

Alternative	Total
NA-1. The primary no-action plan: Bakers Haulover Inlet ebb shoal borrow area	$6,576,000
NA-1. The primary no-action plan: Dade deepwater sites	$6,659,000
S-1. Rehabilitation of existing groins.	$1,842,000
S-2. Construction of new groin field.	$2,265,000
S-3. Construction of tuned groin field.	$2,768,000
S-4. Construction of offshore breakwaters.	$3,637,000
S-6. Extension of Bakers Haulover Inlet south jetty.	$8,661,000
S-7. Construction of sand bypassing facility. See note 1.	$4,705,000
B-1. Construction of historic beach fill. Bakers Haulover Inlet ebb shoal borrow area	$6,576,000
B-1. Construction of historic beach fill. Dade deepwater sites	$6,659,000
B-3. Construction of feeder beach. Bakers Haulover Inlet ebb shoal borrow area	$9,277,000
B-3. Construction of feeder beach. Dade deepwater sites	$9,393,000
B-4. Construction of nearshore berm. Bakers Haulover Inlet ebb shoal borrow area	$9,277,000
B-4. Construction of nearshore berm. Dade deepwater sites	$9,393,000

Note 1. Estimated annual operating cost is $266,500. Estimated annual capital replacement overhaul cost is $12,696, for a total estimated annual cost of ~$279,200.

The costs of alternatives presented in table 19 were estimated using unit price methodology in keeping with Engineering Regulation 1110-2-1302, Appendix C, Page C-2. The unit prices used to develop construction costs for each of the alternatives were taken from Bid Award sheets for recently awarded construction projects of similar types, in similar locations. Preconstruction engineering and design (PED) and supervision and administration (S&A) were applied at eight and ten percent, respectively. Contingency was applied at 25 percent.

Economic evaluations will be based on a 50-year period of analysis. This 50-year period began with initial construction of the Bal Harbour segment of the Dade County BEC & HP project in 1975, and the economic analysis will therefore extend through the year 2025. The baseline year for the economic analysis presented in this report will be 2004, which corresponds to year 29 in the 50-year period of analysis. It will be assumed that construction of the proposed improvements will be performed in 2009, at the time of the next projected renourishment of Bal Harbour.

This economic analysis will be based on a straightforward comparison of annualized project costs through the remaining 21 years of the project's economic life. The average annual equivalent cost of maintaining the project in its current configuration (plan NA-1) will be compared to the annual costs of maintaining the project under the remaining alternative plans of improvement. This procedure will determine the most economically efficient method of maintaining the Bal Harbour segment of the project through the remaining 21 years of the project life.

Based on the expected shoaling rates from the sediment budget presented in this report, the Bakers Haulover Inlet ebb shoal may be re-used as a borrow source for every second or every third renourishment of the Bal Harbour shoreline. The ebb shoal was used as a borrow source for the 2003 renourishment of Bal Harbour, and monitoring of the infilling rate of the borrow pit will be used to calculate the allowable frequency of usage of this borrow site for future renourishments. Due to the uncertainty as to the availability of the ebb shoal borrow site, two analyses will be presented in this discussion : in the first case the ebb shoal will be assumed to be available for every other renourishment. In the second case the ebb shoal will be assumed to be available for every third renourishment. Since the estimated cost for using the ebb shoal is only slightly lower than the cost for using the deepwater site, the frequency of availability of the ebb shoal borrow site does not strongly influence this economic analysis.

Economic Evaluation. Each structural alternative would be constructed in combination with periodic beach renourishment, except for alternative S-7, the sand bypassing facility. Three configurations of beach fill placement are included in the final array of alternative plans : alternatives B-1 (historic fill template), B-3 (feeder beach), and B-4 (nearshore berm). In order to reduce the number of combinations of beach fill configurations with structural improvements, the beach fill alternatives will be evaluated first, as stand-alone projects.

Construction costs for the three beach fill configurations are presented in tables 19 and 20. Renourishment intervals and average annual equivalent costs over the remaining 21 years of the project life are provided in table 20 below. Note that the beach fill design for the no-action plan (NA-1) is identical to the construction of the historic fill template B-1. Similarly, the volumes of fill placement are the same for alternatives B-3 and B-4; only the configuration of the beach fill differs. Estimated average annual equivalent costs are provided in table 20 for each beach fill alternative based on two scenarios : using the ebb shoal borrow site for every second renourishment, and using the ebb shoal borrow site for every third renourishment.

Table 20 - Annualized Renourishment Costs, Beach Fills

Alternative	Construction Cost, ebb shoal	Construction Cost, deepwater	Renourishment Interval	AAE Cost 1	AAE Cost 2
NA-1	6,576,000	6,659,000	6	929,926	930,974
B-1	6,576,000	6,659,000	6	929,926	930,974
B-3	9,277,000	9,393,000	6.75	1,270,909	1,272,460
B-4	9,277,000	9,393,000	6.75	1,270,909	1,272,460

1) Assuming Bakers Haulover Inlet ebb shoal borrow area used every 2nd renourishment
2) Assuming Bakers Haulover Inlet ebb shoal borrow area used every 3rd renourishment

As seen in table 20, continued construction of the historic beach fill template is clearly more cost-efficient than either of the modified beach fill designs presented in alternatives B-3 and B-4. The longer renourishment interval of alternatives B-3 and B-4 are more than offset by the higher construction costs associated with constructing the larger and more complex feeder beach template (B-3) and nearshore berm (B-4). Based on the preceding analysis, construction of the renourishment template as described in alternative B-1 throughout the remaining life of the project will result in an average annual equivalent cost of $929,926, compared to $1,270,909 for constructing the fill as described in alternatives B-3 and B-4. Selecting alternative B-1 therefore results in an annual cost savings of $340,983 over alternatives B-3 and B-4, based on using the ebb shoal borrow area every 2nd renourishment. The cost differential between alternative B-1 and alternatives B-3 and B-4 becomes slightly greater when the deepwater site is used more frequently. If the ebb shoal is used as a borrow source for every 3rd renourishment, the annual cost for alternative B-1 becomes $930,974, compared to $1,272,460 for constructing alternatives B-3 and B-4. This results in an annual savings of $341,486 for future project renourishments.

GENESIS numerical modeling indicated that construction of either alternative B-3 or B-4 in combination with various structural alternatives did not significantly increase projected project performance when compared to beach fill alternative B-1 in combination with the same structural improvements. Due to the cost savings and proven historical performance, beach fill alternative B-1 will be selected as the recommended design for future periodic renourishments of the Bal Harbour shoreline.

The average annual equivalent costs of the proposed structural improvements will now be evaluated, with periodic renourishments based on the design presented in alternative B-1. Table 21 contains a cost analysis matrix for the remaining structural alternatives, in a format similar to table 20. This table includes construction costs, renourishment intervals, and average annual equivalent costs based on the same two renourishment scenarios presented in table 20 : i.e. use of the ebb shoal borrow site for every second renourishment, and use of the ebb shoal borrow site for every third renourishment.

Table 21 - Annualized Renourishment Costs, Structural Improvements

Alternative	Structure Cost	Annual O&M Cost	**Renourish Cost,**ebb shl	**Renourish Cost,**deepwater	Renourish Interval	AAE Cost 1	AAE Cost 2
NA-1	0	0	6,576,000	6,659,000	6.00	929,926	930,974
S-1	1,842,000	1,000	6,576,000	6,659,000	6.50	1,026,771	1,027,861
S-2	2,265,000	1,000	6,576,000	6,659,000	6.80	1,041,951	1,043,064
S-3	2,768,000	1,000	6,576,000	6,659,000	8.50	850,094	853,359
S-4	3,637,000	5,000	6,576,000	6,659,000	7.25	1,111,453	1,112,598
S-6	8,661,000	1,000	6,576,000	6,659,000	7.00	1,435,162	1,436,290
S-7	4,705,000	279,200	0	0	N/A	991,746	991,746

1) Assuming Bakers Haulover Inlet ebb shoal borrow area used every 2nd renourishment

2) Assuming Bakers Haulover Inlet ebb shoal borrow area used every 3rd renourishment

As seen in tables 20 and 21, the baseline cost for comparison of average annual equivalent costs is \$929,926 (ebb shoal every 2^{nd} renourishment) or \$930,974 (ebb shoal every 3^{rd} renourishment). These are the annual costs of the no-action plan (NA-1) against which all costs of structural improvements will be measured. As seen the right-hand columns of table 21, average annual equivalent costs are computed for each structural alternative. These costs include an amortization of the construction costs of the proposed structure in the year 2009, at the time of the next projected renourishment of the Bal Harbour shoreline. Renourishment costs are calculated for each alternative based on the projected renourishment intervals obtained from GENESIS numerical shoreline modeling. Annual operation and maintenance (O&M) costs are also included table 21; these typically consist of the costs of structural monitoring and minor repairs, with the exception of alternative S-7. The AAE costs in the right-hand columns of table 21 represent the total of amortized construction costs, periodic renourishment costs, and maintenance costs.

Based on this analysis, alternative S-3 (construction of T-head tuned groin field) provides the lowest average annual equivalent project cost throughout the remaining 21 years of the project's 50-year period of economic analysis. This cost is substantially lower than the annual costs of the other alternative plans, primarily because the longer renourishment interval of alternative S-3 results in only two future renourishments throughout the remaining 21 years of the project; whereas the remaining plans each require three renourishments. A comparison of the annual cost of alternative S-3 also shows that the cost of the proposed improvement is less than the cost of maintaining the project in its existing configuration (NA-1).

Based on the preceding economic analysis, as well as environmental and aesthetic considerations, and the wishes of the project's local sponsor, alternative S-3 is selected as the optimum plan of structural improvement for the Bal Harbour segment of the Dade County BEC & HP project. The project will continue to be renourished using the historical beach fill template as described in alternative B-1.

DETAILED PROJECT DESIGN

Summary of Physical Processes.

Erosion along the Bal Harbour shoreline has been shown to be due to a combination of factors including the interruption of sediment transport along the shoreline created by Bakers Haulover Inlet, some degree of wave energy focusing caused by the irregular offshore bathymetry, and the presence of a nodal point from which sediment is transported in both directions. A thorough understanding of these processes is essential to selecting the appropriate plan of improvement for reducing erosion along the Bal Harbour shoreline.

As discussed in previous sections of this report, the net littoral transport potential averages about 60,000 cy/yr along most of northern Dade County's shoreline, but only about 19,000 cy/yr bypasses the littoral barrier created by Bakers Haulover Inlet. This littoral deficit is directly responsible for a large portion of the erosion observed along the Bal Harbour shoreline. As discussed previously, the wave climate along the Dade County coast is bimodal; littoral transport is generally directed toward the south in the winter and toward the north in the summer. The effect of this process is to accumulate sand along the northernmost section of Bal Harbour during the summer months as sand travels northward. The accumulation begins at the Bakers Haulover Inlet south jetty, and spreads further southward during the summer months. During the winter months this process is reversed. The blockage of sediment caused by the inlet affects the northernmost portion of Bal Harbour first as sediment is eroded from immediately south of the south jetty. Over time the eroded area spreads further southward along the length of Bal Harbour.

As shown previously in the STWAVE analysis, some wave energy focusing due to offshore bathymetry and tidal currents also contributes to erosional losses along this reach of shoreline. Refraction of incoming waves around the Bakers Haulover Inlet ebb shoal creates a zone of higher wave energy in the lee of the shoal. This wave energy focusing is directed more along the southern shoreline of Haulover Park, but some focusing along the northern 2,000 feet of Bal Harbour is also apparent. This increase in breaking wave height translates to increased sediment transport, and increased shoreline erosion. Breaking wave angles are likewise modified by the refraction process; the more oblique wave angles resulting from the refraction process also tend to increase the erosive potential of waves in these regions near the inlet.

The presence of a nodal point is also apparent from the STWAVE analysis. This point can range from 600 to 1,100 feet south of the Bakers Haulover Inlet south jetty for waves originating from the easterly and northeasterly directions. For waves originating from southerly directions no nodal point exists. Erosion along the northern reach of the Bal Harbour segment is accelerated by the presence of this nodal point, as sediment is transported out of the area in both directions.

An array of alternative plans of improvement was developed in preceding sections of this report. These alternatives were evaluated for effectiveness in reducing erosion along the Bal Harbour shoreline in accordance with the physical processes described above. Other factors such as environmental impacts, construction and maintenance costs, pubic safety, and aesthetics were considered in choosing the recommended plan of improvement. The plan that best fulfilled these project requirements was option S-3, (construction of T-head rubble-mound groin field) in combination with option B-1 (continued renourishment of Bal Harbour using the current beachfill design). Detailed design of the recommended plan is described below.

<u>Detailed Design of Recommended Plan.</u>
General Description of Plan. The recommended plan of improvement along the Bal Harbour shoreline was developed based on a study of the physical processes along the study area and fine-tuned using the numerical shoreline change model GENESIS. The recommended plan consists of a combination of elements, including reconstruction of the existing groin field, some structural improvements to the existing groin field, and continued periodic beach renourishment of the Bal Harbor project reach. For simplicity throughout this report the groins are numbered 1 through 5 proceeding from north to south along the Bal Harbour shoreline. The upland portion of the berm throughout Bal Harbour consists of an extensively vegetated public park, and neither the park nor the associated vegetation will be disturbed during the construction of any elements of this plan, as per the local sponsor’s wishes. For all practical purposes the baseline for construction of this project will be the seaward edge of the vegetation line along the length of this park.

Design of Rubble-Mound Groin Field. The main element of the proposed plan of improvement is the replacement of the existing king pile groin field. The existing king pile groins will be removed and reconstructed using rubble-mound structures. These rubble structures will be less permeable and will reflect less wave energy and induce less localized scouring than the vertical king piles. The new rubble-mound structures will be built over the top of existing groins 1,2,3, and 5. In order to achieve a more uniform spacing throughout the groin field, groin 4 will be removed entirely and reconstructed 100 feet to the south of its present location. This will result in an average groin spacing of about 850 feet.

Several modifications will be incorporated into the new groin field design to improve project performance. T-heads will be added to groins 1 and 2 to increase the structures' effectiveness at reducing losses along the northern portion of the project. Groin 3 will be rebuilt to the same dimensions as the original king pile structure. The king piles of groin 4 will be completely removed, and the structure moved 100 feet further south to provide a more even spacing along the groin field. Groins 4 and 5 will each be shortened from their present lengths to provide a taper along the southern portion of the groin field, to reduce downdrift effects. This combination of T-heads, groin lengths, and groin spacing provided a much improved shoreline response over the existing conditions in the GENESIS modeling.

Groin 1 lies within the area of tidal influence of Bakers Haulover Inlet, and as a result, sediment movement tends to be directed toward the north (into the inlet) much of the time along this reach of shoreline. Results from the wave transformation model STWAVE indicate that waves refracted by the inlet's large ebb shoal are directed toward the inlet, and rarely create southward sediment transport along the shoreline north of the position of groin 1. Under existing conditions, sediment north of this position appears to be driven into the inlet by this refraction-induced littoral transport, and is further aided by both ebb and flood tidal currents.

Flood tidal currents flow directly toward the inlet. Although flood currents velocities directly along the shore are low, they still tend to increase the longshore current velocities and therefore the transport of sediment northward along the beach and into the inlet. Ebb tidal currents tend to increase transport toward the inlet by two processes. First, strong ebb flows through the inlet tend to create large vortices (or eddies) which can reach the shoreline. South of the inlet these currents rotate clockwise on the ebb flow, and when in close proximity to the beach can transport sediment northward toward the inlet. Secondly, ebb tidal currents flowing into the open ocean refract incoming waves to a higher degree, causing waves to strike the shoreline near the inlet at a much steeper angle (oriented toward the inlet), again further increasing northward sediment transport along the northern Bal Harbour shoreline.

In order to stabilize this reach of the Bal Harbour shoreline against the northward sediment transport from the physical processes described above, the following modifications of the northern two groins are recommended. Groins 1 and 2 will be reconstructed along their present alignments to the post-nourishment mean high water line (approximately their present lengths). T-heads will then be added to the seaward ends of both structures to reduce the losses of sediment from within the shoreline cells extending between the Bakers Haulover Inlet south jetty and groin 1, and between groins 1 and 2. Both T segments on groin 2 and the south T segment on groin 1 will be oriented roughly shore-parallel, and each will extend 25 feet outward from the centerline of each groin. The north T segment on groin 1 will be oriented approximately 10 degrees to the east of shore-parallel, and will extend outward 50 feet from the structure's centerline. The orientations of all T-head segments were chosen to lie perpendicular to the predominant direction of incoming wave energy for maximum effectiveness.

The ebb shoal at Bakers Haulover Inlet provides a pathway for natural transport of sediment around the inlet. As discussed throughout this report, a primary objective is to allow maximum natural bypassing of sediment to continue along this shoal. Any structural plan which significantly interferes with the predominantly southward transport of sediment along this shoal will likely result in increased erosion south of the inlet, corresponding to higher renourishment rates (and costs). The ebb shoal re-connects to the Bal Harbour shoreline in the vicinity of groin 2, and sediment transport from the shoal to the beach appears to occur along most of the remaining reach of Bal Harbour to the south. In order to avoid any interruption of sediment flow along the landward region of the shoal and to minimize downdrift effects due to disruption of littoral transport, the remaining groins to the south (groins 3, 4, and 5) will be reconstructed without the T-head segments. Furthermore, as an added measure to prevent downdrift erosion caused by excessive impoundment of sand behind the structures, groins 4 and 5 will be tapered in length to allow increasing amounts of bypassing near the southern limit of the project.

Field data and design guidance suggest that a 6-degree plan-view taper is effective at reducing erosion at the transition between a groin field and the adjacent natural beach, while still maintaining adequate beach widths within the groin field (CETN-III-12, reference 3b). This criteria was applied in several configurations during GENESIS modeling; the most favorable shoreline response occurred when groin 3 was kept at its full length and groins 4 and 5 were shortened according to the 6-degree criteria. This configuration will be adopted for the recommended plan.

Construction of Rubble-Mound Groin Field. The new rubble-mound groins 1, 2, 3, and 5 will each be built over the corresponding king pile structure. Groins 1, 2, and 3 will each be rebuilt to the same approximate length as the original structures. Groins 4 and 5 will each be shorter than the original king pile groins. Groin 4 will be moved 100 feet to the south of its present position. T-heads will be added to the seaward ends of groins 1 and 2 to further reduce erosional losses from the northern 2,000 feet of Bal Harbour, which is the most erosive area of the project. The king piles and horizontal connecting panels along each of the five original groins will be removed, from the seaward ends of the structures to the vegetation line. A plan view of the limits of groin reconstruction and beach renourishment is shown in figure 29.

The king piles were constructed on 10-foot centers, and removal of approximately 95 piles will be required prior to reconstruction of the five rubble groins. It is not known how many of the horizontal panels between the king piles may still be in place (at this time all five king pile groins are completely buried from the 2003 Bal Harbour renourishment). In addition, scattered rubble is visible at the seaward end of each king pile groin, and may extend along the length of the structures. Any rubble or horizontal panels which are encountered during excavation of the new groins' foundations will be removed. All king piles, concrete panels, and rubble will be stockpiled off-site for use by the local sponsor as reef-building material.

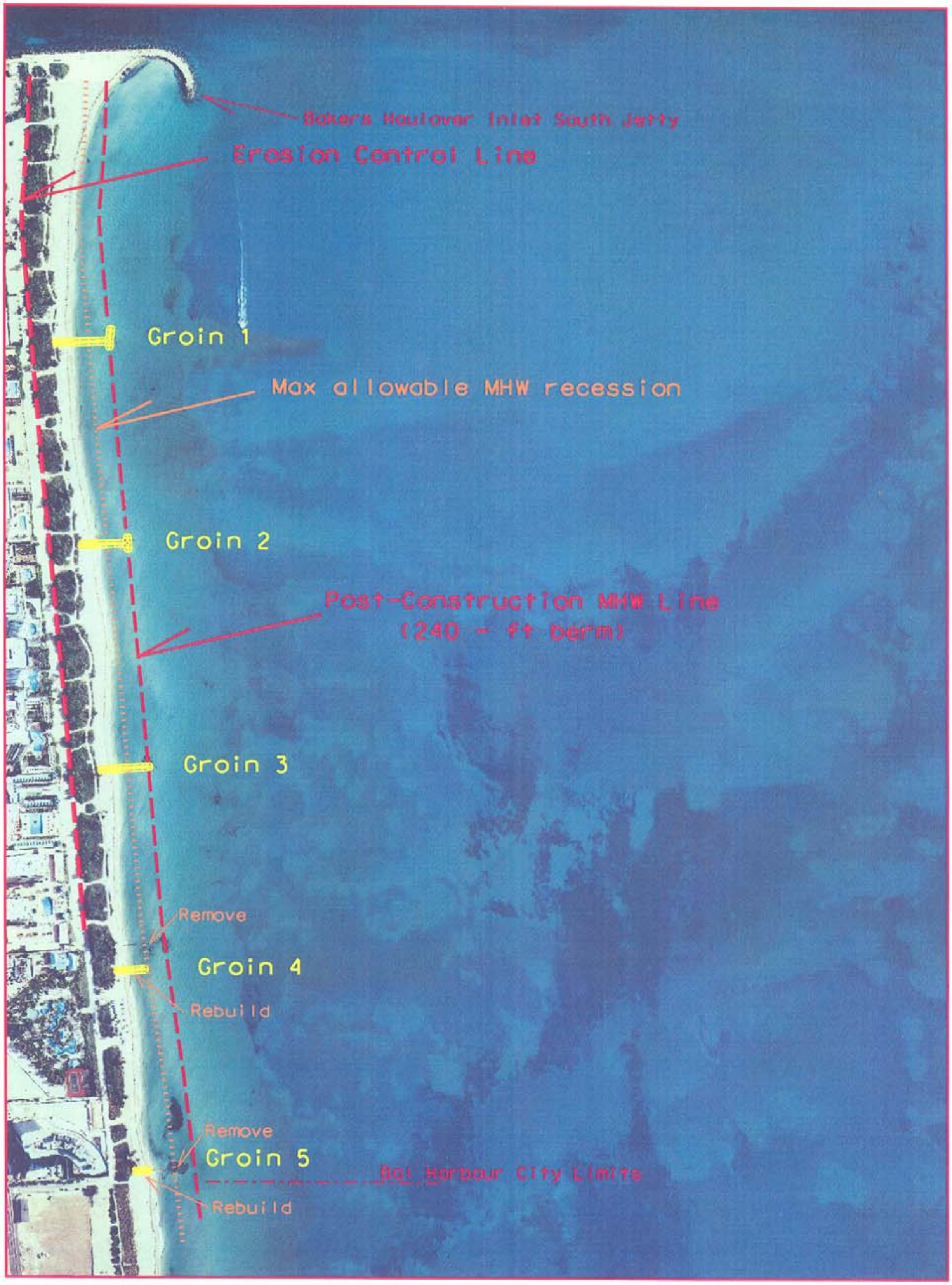


Figure 29. Recommended Plan of Improvement – Plan View.

Little information is available on the design of the existing king piles, particularly the depth of embedment. Most of the king piles remain buried in the beach fill and even the top elevations of those piles are not known. Therefore either of two methods of construction will be acceptable, depending on the practicality of removing the piles. The piles should be removed intact if possible but if the depth of embedment is such that pile extraction is not reasonably practical, piles may be cut as follows : the piles which will be covered by new groin construction may be cut at elevation +1 foot, mlw; the piles which fall outside of the footprint of the new rubble groins may be cut at elevation –5 feet, mlw. The deeper cut-off elevation for piles outside of the footprint is required for public safety as well as environmental and aesthetic reasons. Similarly, the required depths for removal of the existing rubble and concrete panels is as follows. For areas within the footprint of the new rubble-mound groins, rubble and panels must be excavated to the base elevation of the foundation, -3 feet mlw. For all areas outside of the footprint of new groin construction, these materials must be excavated to a minimum depth of –5 feet mlw.

The above specifications for removal of the existing groins will require that all king piles be removed along the full lengths of groins 1, 2, and 3 from the vegetation line to the seaward end of the structures. If these piles cannot be removed they may be excavated and cut at elevation +1 foot mlw. All panels and rubble would be excavated to elevation –3 feet mlw along the lengths of the structures. Any materials outside of the footprint of new construction would be removed to an elevation of –5 ft mlw. Groin 4 is to be completely removed and rebuilt 100 feet to the south. Therefore king piles along the length of groin 4 would be removed completely or cut at elevation –5 ft mlw; rubble and panels would also be removed to the –5 ft mlw elevation. Groin 5 is to be reconstructed along the existing alignment, but to a shorter length. King piles along the alignment of groin 5 would therefore be removed or cut to an elevation of +1 ft mlw (inside new groin footprint) or to elevation –5 ft mlw (outside of new groin footprint). Rubble and panels along groin 5 would be removed to –3 ft mlw (inside footprint) or to –5 ft mlw (outside footprint).

The northern three groins (groins 1, 2, 3) will extend from the vegetation line seaward to the post-renourishment mean high water line. Based on the 240-foot construction berm and an equilibrium front slope of 1v:11h, a distance of 305 feet is calculated from the ECL to the mhw line. The vegetation line is located well seaward (80 to 110 feet) of the ECL along the length of Bal Harbour. Since the local sponsor does not wish to destroy the exotic upland vegetation and other facilities within the park, the existing king pile groins will remain in place under the vegetated upland portion of the beach and no construction will extend into the vegetated areas. Maintenance of the authorized design berm will assure that adequate material remains in place to prevent flanking of any of the reconstructed groins.

The measured lengths of king pile groins to be removed and rubble-mound groins to be rebuilt are tabulated in table 22. All distances are measured from recent controlled aerial photography, and represent centerline distances from the seaward edge of vegetation to the seaward end of the specified structure. Based on these measurements the following lengths of groin removal and reconstruction will be required :

Table 22
Lengths of Groin Removal and Reconstruction – Recommended Plan

	Length Removed[1]	Length Rebuilt[1]
Groin 1	220	220 (295)[2]
Groin 2	190	190 (240)[2]
Groin 3	200	200
Groin 4	205	120
Groin 5	215	60

[1] Measured from vegetation line (80-110 feet seaward of ECL)
[2] Including T-head sections

The required armor stone size is calculated based on the maximum depth-limited wave which can impact the structures. The maximum breaking wave height at each groin is limited by the maximum water depth seaward of the structure. To determine the largest wave which can reasonably be expected to impact the groins, all available surveys of the area were examined to determine the maximum measured water depths near the seaward tips of the five groins. An examination of recent survey data indicates that the 1996 pre-fill survey provides the most comprehensive view of the project in a fully-eroded condition. In this survey the maximum mean low water depth at the seaward tips the groins is about –2 to -3 feet. Due to the variability of the nearshore depths and in the interests of providing a successful design under the entire range of bathymetric conditions, a maximum mlw depth of –3.0 feet will be assumed for the design of each of the five rebuilt groins. Due to similar site conditions at each groin location, the same stone size will be specified for each structure. This will allow for easier and more economical construction of the groins as well.

The maximum depth at the seaward tip of the structures is dependent on water levels as well as the maximum scour depth described above. According to data presented previously in this report (see table 1 and figure 5), the normal tide range in this area is 2.54 feet, and surge levels associated with hurricanes of various intensities are much higher. Maximum impact forces on the structure will occur with the water level near the crest of the groin. Based on the design crest elevation of +4 feet mlw a maximum water depth of 7 feet is calculated, which corresponds to a 'northeaster' storm surge with a return interval of about 5 years. Since maximum depth-limited wave height is calculated as 0.78 * water depth, a maximum design wave height of 5.5 feet is calculated. Higher water levels can support higher incident waves, but the structure would be protected to a large degree by the overlying water.

From Hudson's equation, based on the 5.5-foot design wave, a median armor stone size of 1.2 tons is calculated. This calculation is based on structure side slopes of 1v : 1.5h and a stone density of 165 lb/cubic foot. The 1v : 1.5h side slope is considered necessary to achieve the desired high porosity of the groins; this is the steepest stable slope which can be constructed and represents the narrowest possible rubble structure at the base. The 165 pcf density is specified because as stone density decreases, the required armor stone size increases. For example, for the 5.5-foot design wave the required median armor stone weight is 1.2 tons for 165 pcf stone, but increases to 2.4 tons for 140 pcf stone (typical density for native Florida limestone). In this case it is desirable to keep the cross-section of the groins to a minimum, so the armor stone size must be minimized as well, and 165 pcf stone is specified.

The cross-section of each groin will be identical, and is described as follows. The crest elevation of 4 feet mlw will allow bypassing of sediment over and through the structures. The crest of each groin will be three stones wide (7.5 feet) as per CEM recommendations (reference 3n). The foundation of each groin will be constructed at –3 feet mlw, which coincides with the maximum expected depth of scour around the structures. A bedding layer 1 foot thick will be constructed under the armor layer using marine mattresses, and the mattresses will extend 5 feet beyond the limits of the armor stone for scour protection.

No intermediate or core stone will be used. Woven geotextile fabric will be placed under the foundation mattresses. The T-sections on groins 1 and 2 will also be constructed using this design cross section. A cross-section of the proposed design is shown in figure 30.

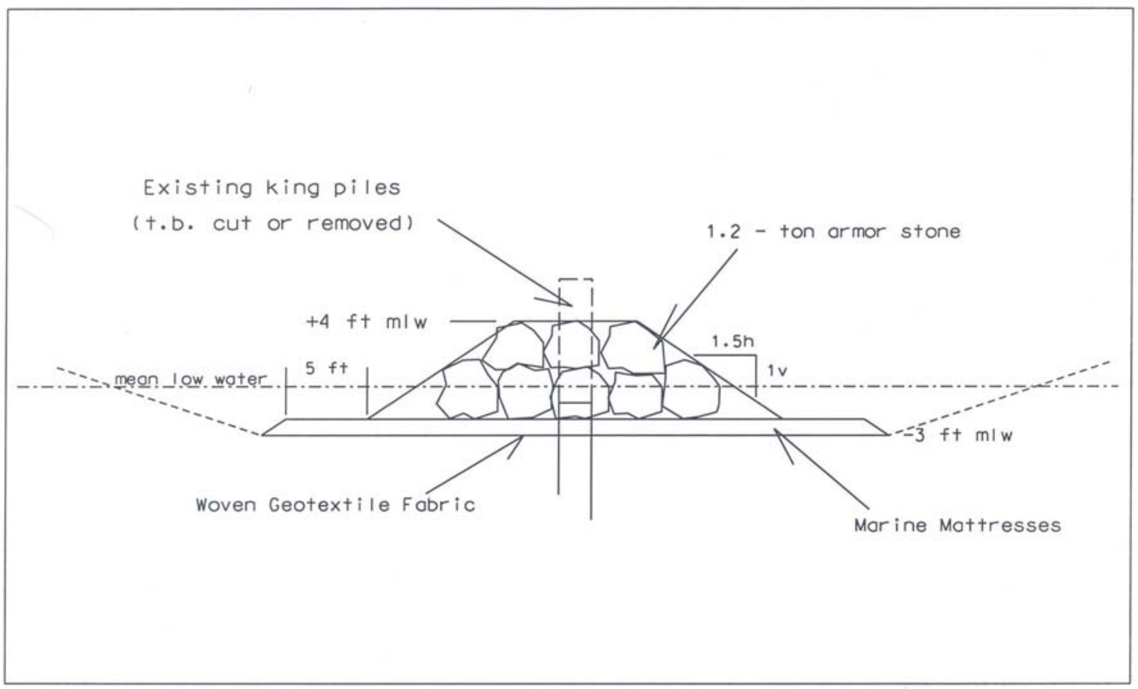


Figure 30. Recommended Plan – Cross Section.

Removal of the five existing king pile groins and reconstruction of the five modified groins could be accomplished using either of two methods :

a) The existing structures could be removed and new groins constructed by barge-based equipment or by construction of temporary sand ramps while the beach is in a fully-eroded condition prior to the next beach renourishment. Construction materials could be transported to the site by barge or by land. Some land-based equipment and stockpiling of materials would still be required to construct the upland portions of each of the five groins. This operation could be conducted only during periods of calm weather.

b) The existing structures could be removed and reconstructed from land immediately following the next beach renourishment. Since each of the five groins extends to a maximum seaward distance of the post-renourishment mean high water line, construction could be conducted entirely on land if the groin reconstruction followed the next beach renourishment. Excavation across the 240-foot wide berm would be required, and all materials and equipment would be stockpiled on site. This method of construction would be much less dependent on weather than the barge-based construction method, but would require much greater quantities of excavation.

The contractor would be allowed to choose either construction method, or a combination of both methods. For example, the contractor may choose to remove the existing king pile groins while the beach is in the fully eroded (pre-renourishment) condition, then construct the new structures after placement of the renourished beach. Since only the northern three groins extend to the mhw line, and since significant erosion of the beach fill is not expected during the calm summer months when renourishment is typically performed, erosional losses of the beach fill during groin construction are expected to be minimal.

Periodic Renourishment. Based on the preceding volumetric analysis, the annual erosion rate along the length of Bal Harbour has averaged –55,600 cy/yr between 1990 and 2002. The volume required to build the 240-foot construction berm during the past three project renourishments has averaged about 330,000 cubic yards, and these renourishments have been required an average of every 8.3 years. The northern portion of the project (R-27 to R-29, or between the south jetty and groin 2) has historically eroded much more rapidly than the southern reach of the project. The main functions of the redesigned groins are to reduce and more uniformly distribute these erosional losses along the length of Bal Harbour.

The renourishment plan would consist of periodic rebuilding of the same 240-foot construction berm which has been constructed since initial construction of the Federal project. A berm narrower than 240 feet would result in less advanced nourishment being placed and would only increase the frequency of renourishment. A wider berm would experience accelerated erosion due to the steeper nearshore profile, and would likely experience very high losses at the north end of the fill due to tidal current scouring. A wider berm would extend beyond the protective jetty and would also cover a larger footprint, possibly including nearshore hardbottom. The potential coverage of hardbottom would require extensive permitting and/or mitigation. GENESIS modeling runs indicate that the proposed modification of the groins would more effectively hold the 240-foot berm in place, resulting in an increased renourishment interval.

At the present time all of Dade County’s originally-permitted offshore borrow sites have been depleted, except for a small portion of the SCG borrow area, located south of Government Cut. This area is being reserved exclusively as a source of emergency beach fill, and there are no remaining fully-permitted offshore borrow areas available for use. The ebb shoal at Bakers Haulover Inlet was used as a borrow source for the 2003 Bal Harbour renourishment. The plan for dredging this shoal was designed to minimize impacts to natural sediment bypassing around the inlet by maintaining the general shape of the shoal.

In order to maintain the present degree of bypassing the general shape of the shoal must be maintained during any future dredging events, and excessive dredging of this shoal must be avoided to prevent detrimental impacts to natural sediment bypassing around the inlet.

The Bakers Haulover Inlet ebb shoal is a naturally-accreting feature, and may be used again in the future when the shoal recovers from the 2003 dredging. Periodic use of this shoal as a long-term borrow site for Bal Harbour is recommended, as it essentially amounts to mechanical bypassing of sediment around Bakers Haulover Inlet and mimics the natural bypassing process. The infilling of the borrow area dredged in 2003 will be monitored to determine how often this region can be used as a source of borrow material. Based on the accretion rate of 32,000 cy/yr presented in the regional sediment budget, it is currently estimated that the Bakers Haulover Inlet ebb shoal could be used as a source of sediment for Bal Harbour during every second or every third renourishment.

Other sources of sediment are detailed in the Dade County Evaluation Report (reference 1g). These sources include material remaining in the original offshore borrow areas, deepwater sites offshore of Dade County, borrow sites off Palm Beach, Martin, and St Lucie Counties, upland borrow sites, carbonates/aragonite, and sand relocation from the South Beach area. Technical information on each of these sites is presented in the Geotechnical Appendix (Appendix B) following the main text of this report, and in reference 1g.

All of these sites remain available except for the carbonate/aragonite and sand relocation options. The District is prohibited from using non-domestic sources such as carbonates/ aragonite by a directive from Headquarters. The relocation of sand from South Beach has been shown in the Evaluation Report to be a valid option only for reaches of the project along the southernmost portion of the Federal project; Bal Harbour is located too far north for this source to be economical or feasible. Any of the remaining sources of material could be used to supply sediment for future renourishments. A generic sand specification has been developed by the Geotechnical Branch of the Jacksonville District, US Army Corps of Engineers, for use on the Dade County project. This specification dictates the properties of sediment that can be used for renourishment of the Dade County BEC & HP project, and any material that meets the criteria of this specification could be used for renourishment of any segment of the project, including Bal Harbour.

The deepwater site located along the edge of the continental shelf along the Dade County shoreline appears to be one of the most favorable options. Sand deposits in excess of 20 million cubic yards have been discovered in water depths ranging from 60 to over 250 feet along this region. The continental shelf is only 2 to 3 miles wide along Dade County, so transport distance from the borrow area to the fill area is minimal. While this is deeper than conventional dredging depths, existing equipment can be modified to excavate this borrow material. Due to the large volume of material contained in this potential borrow site, all segments of the Dade County BEC & HP project could be maintained throughout the remaining years of the project life if this borrow area was developed for use. Because of the relatively high cost of mobilizing and/or modifying a dredge specifically for deep-water use, it is recommended that renourishment schedules be coordinated county-wide to take advantage of each deepwater dredging event. Currently, further geotechnical investigations are required to define the extent of the usable area and the volume and quality of the material contained within it. Permitting of this borrow area is also required prior to its use.

The most likely scenario for future maintenance of the Bal Harbour segment of the Dade County BEC & HP project involves rotating between the two most favorable borrow areas : the Bakers Haulover Inlet ebb shoal and the deepwater sites along the continental shelf. Depending on the infilling rate, it is estimated that the ebb shoal could be used during every second or every third renourishment of Bal Harbour. The more expensive deepwater site would be used during times when the ebb shoal was not available for use.

In accordance with Engineering Regulation ER 1110-2-1302, an MCACES cost estimate was prepared for the recommended plan using labor/equipment/materials-type formatting. This cost estimate is provided in its entirety in Appendix D. The work breakdown structure feature and subfeature levels of detail are shown on the MCACES printout, as required by the Engineer Regulation. As was the case with the initial set of cost estimates performed for the screening of alternative plans (tables 19, 20, 21), preconstruction engineering and design and supervision and administration were applied at eight and ten percent, respectively, and contingency was applied at 25 percent. The changes in costs from the alternative screening level to the MCACES/recommended plan level is due to the change in cost methodology from unit price methodology to labor/equipment/materials methodology.

Based on the detailed MCACES estimates presented in Appendix D (updated to October 2005 price levels), the cost of rebuilding the king pile groin field is $ 2,724,000. The cost of reconstructing the 240-foot berm along the length of Bal Harbour using the ebb shoal borrow area is $ 6,270,000; using the deepwater borrow area the cost is $ 7,201,000. The Bakers Haulover Inlet ebb shoal was used as a source of material for the recent 2003 renourishment of Bal Harbour, and it is expected that this area will not be available for use as a borrow area for construction of the proposed improvements in 2009. Based on use of the Dade County deepwater borrow sites, the total construction cost of the groin field and reconstructed 240-foot berm as described in this report will be $ 9,925,000.

ENVIRONMENTAL DOCUMENTATION

No significant adverse environmental impacts are expected from the actions recommended in this report. The Environmental Assessment, the Finding of No Significant Impact, and other related documents are provided in Appendix A of this report.

REAL ESTATE

Construction of the beach fill and all structural improvements as recommended in this report are located seaward of the Erosion Control Line (ECL) and fall within State of Florida ownership. No additional lands are required.

CONCLUSIONS

This Detailed Design Report examined the performance of the entire Dade County Beach Erosion Control & Hurricane Protection project during the twelve-year period from 1990 to 2002. This interval extends roughly from the time of completion of the final segment of the Federal project at Sunny Isles in 1988 to the most recent county-wide lidar survey, performed in November 2002. Nearshore processes were examined in accordance with regional sediment management policies. Based on this regional analysis several erosional hotspots were identified, including the erosive area along northern Bal Harbour.

Erosion along the Bal Harbour shoreline has been shown to be due to a combination of factors, including the interruption of sediment transport along the shoreline created by Bakers Haulover Inlet, some degree of wave energy focusing caused by the irregular offshore bathymetry and tidal currents, and the presence of a nodal point along the shoreline from which sediment is transported in both directions. Several alternative plans of improvement were developed with the objective of reducing coastal erosion caused by the processes described above. The alternative plans of improvement developed in this report included the use of stabilizing structures, different beach fill configurations, and innovative technologies.

The design of each alternative plan of improvement was developed based on an examination of physical processes within the study area, and refined using the shoreline simulation model GENESIS. Each alternative was evaluated based on a number of criteria including project performance, environmental compatibility, cost, public safety, and aesthetics. The recommended plan of improvement was selected based on a combination of these factors. The recommended plan was shown to provide a more economically efficient operation of the Bal Harbour segment of the Federal BEC & HP project through the remaining 21 years of the project’s 50-year period of economic analysis.

The recommended plan of improvement as developed in this study consists of the removal of the five existing king pile groins and reconstruction of five rubble-mound groins in an improved configuration. The northern two groins will be constructed with T-heads to increase project performance, while the southern two groins will be tapered in length to reduce impacts to downdrift shorelines. The project shoreline will continue to be renourished using the same 240-foot construction template which has been used since initial construction of the project.

This report also addressed the future maintenance requirements of the project, particularly the sources of borrow material needed for future renourishments of the project throughout the remaining years of the 50-year project life. All but one of the existing offshore borrow sources have been depleted along Dade County, and this report presents data on several alternative borrow sources which may be used to meet future renourishment needs for the Federal BEC & HP project. Potential future sources of fill material include borrow areas located offshore of Dade, Palm Beach, and St Lucie Counties, the use of deep-water sites, upland sand mines, carbonate/aragonite materials, and relocation of beach sediment within the Federal project. Physical data including the characteristics and compatibility of sediment from each site were presented in this report.

At this time the most practical and economically efficient borrow sites for the future maintenance of Bal Harbour appear to be the Bakers Haulover Inlet ebb shoal, and the deepwater borrow sites along the Dade County shoreline. A combination of legal and jurisdictional issues, cost, and practical limitations with dredging and/or transporting material to the fill site may limit use of the remaining borrow sites. As detailed in this report, the use of the ebb shoal borrow area would be allowed only on an occasional basis, as this important natural sediment pathway must be allowed to recover following each use as a borrow site. Based on projected ebb shoal shoaling rates presented in the regional sediment budget it is currently envisioned that the ebb shoal could be used for every second or every third renourishment of the Bal Harbour shoreline. Ongoing monitoring surveys will be used to determine the actual allowable frequency of use of this borrow site. The deepwater borrow site is recommended for use when the ebb shoal is not available.

Detailed MCACES cost estimates were developed for the recommended plan and updated to October 2005 price levels. The cost of rebuilding the king pile groin field is estimated to be $ 2,724,000. The cost of reconstructing the 240-foot berm along the length of Bal Harbour using the Bakers Haulover Inlet ebb shoal borrow area is $ 6,270,000; using the deepwater borrow area the cost is $ 7,201,000. The ebb shoal was recently used as a source of borrow material and is not expected to be available for use at the time of construction. The recommended plan will therefore be based on the use of the deepwater borrow site, with a total cost of $ 9,925,000 for construction of the 240-foot berm and tuned groin field.

RECOMMENDATIONS

It is recommended that the proposed improvements to the Bal Harbour segment of the Dade County Beach Erosion Control and Hurricane Protection project, as described herein, be approved.

Robert Carpenter
Colonel, U. S. Army
District Engineer

REFERENCES

This bibliography is divided into three sections for ease of reference. The first section contains authorization and project design documents produced by the U.S. Army Corps of Engineers which are relevant to the Bal Harbour segment of the Federal project. The second section contains project design and monitoring documents relating to Bal Harbour which were produced by other agencies or private engineering firms. The third section contains general references such as technical papers, design manuals, etc. Documents in each section are listed in chronological order by publishing date.

<u>*Section 1. Corps of Engineers Authorizations & Design Documents:*</u>

1a) House Document 335/90/2.

1b) Dade County Beaches, Florida, Beach Erosion Control and Hurricane Surge Protection Project, General Design Memorandum, U.S. Army Corps of Engineers, Jacksonville District, September 1975.

1c) Beach Erosion Control and Hurricane Protection Study for Dade County, Florida, North of Haulover Beach Park, Survey Report and EIS Supplement, U.S. Army Corps of Engineers, Jacksonville District, June 1982.

1d) Beach Erosion Control and Hurricane Protection, Dade County, Florida, North of Haulover Beach Park, Design Memorandum (CP&E), U.S. Army Corps of Engineers, Jacksonville District, April 1985.

1e) Dade County, Florida, Beach Erosion Control and Hurricane Surge Protection Project, General Design Memorandum, Addendum IV (Nourishment of Beach Segment Between 96th Street to Haulover Inlet), U.S. Army Corps of Engineers, Jacksonville District, September 1987.

1f) Coast of Florida Erosion and Storm Effects Study, Region III, Feasibility Report, U.S. Army Corps of Engineers, October 1996.

1g) Dade County, Florida, Beach Erosion Control and Hurricane Protection Project, Evaluation Report, U.S. Army Corps of Engineers, October 2001.

Section 2. Non-COE Project Design and Monitoring Documents:

2a) Olsen, E.J., and Bodge, K.R., "The Use of Aragonite as an Alternate Source of Beach Fill in Southeast Florida", published in *Coastal Sediments '91, Volume II*, 1991.

2b) Olsen, E.J., and Bodge, K.R., "Beach Nourishment with Aragonite and Tunes Structures", Proceedings of *Coastal Engineering Practice*, ASCE, 1992.

2c) Bakers Haulover Inlet Management Plan, Coastal Planning & Engineering, Inc., March 1995.

2d) Dade County Regional Sediment Budget, Coastal Systems International, Inc, Coral Gables, Florida, January 1997.

2e) Dade County Alternate Sand Source Investigation, Coastal Planning & Engineering, Boca Raton, Florida, September 1997.

2f) Coastal Engineering Report, City of Miami Beach Erosional Hot Spots, Coastal Systems International, Inc, August 1999.

2g) Deep Water Geotechnical Investigation of Offshore Sand Deposits for Beach Renourishment in Dade County, Florida", Coastal Planning and Engineering, and Scientific Environmental Applications, September 2000.

Section 3. General References:

3a) Shore Protection Manual, U.S. Army Corps of Engineers Waterways Experiment Station, 1977 - 84.

3b) CETN-III-12, Groin System Transitions, June 1981.

3c) Flood Insurance Study, Dade County, Florida, and Incorporated Areas, FEMA, November 1987

3d) Effects of Hurricane Andrew on Water Levels in Coastal Florida and Louisiana, Data Report, Technical Memorandum, NOAA, NOS OES 004, December 1992.

3e) Hanson, H. and Kraus, N.C., "Optimization of Beach Fill Transitions", Proceedings from Coastal Zone '93, ASCE, 1993.

3f) Hindcast Wave Information for the U.S. Atlantic Coast, WIS Report 30, U.S. Army Corps of Engineers Waterways Experiment Station, March 1993.

3g) Assessment of Wave Conditions During Hurricane Andrew (1992) at Miami Beach, Florida, U.S. Army Corps of Engineers Waterways Experiment Station, April 1993.

3h) ETL 1110-2-353, Review of Recent Geotextile Coastal Erosion Control Technology, December 1993.

3i) Sorensen, R.M., “Basic Wave Mechanics for Coastal and Ocean Engineers”, 1993.

3j) Lillycrop, W.J., Parson, L.E., and Irish, J.L. (1996), "Development of an Operation on the SHOALS Airborne Lidar Hydrographic Survey System in Laser Remote Sensing of Natural Waters: From Theory to Practice", SPIE 2964, 26-37; 1996.

3k) Surface-Water Modeling System Reference Manual, Brigham Young University Engineering Computer Graphics Laboratory, Brigham Young University, Provo, UT, 1997.

3l) Smith, J.M., Resio, D.T., and Zundel, A.K., "STWAVE : Steady-state Wave Model : Report 1 – Users Manual for STWAVE Version 2.0”, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 1999.

3m) Tide Tables 2001, East Coast of North and South America, NOAA, 2001.

3n) Coastal Engineering Manual, U.S. Army Corps of Engineers, Waterways Experiment Station, 2002.